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Investigation of Closure Pour Elimination for Phased Construction of Steel Girder Bridges

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Abstract

Phased construction is a common practice used by State DOTs during the replacement of a bridge. This method allows for the traffic flow to be maintained on half of the bridge while a new deck is constructed on the other half. For steel girder bridges there is often an issue with differential elevation between the phases. This difference in elevation often prevents the second half of the deck from being poured in one step. Instead, a portion of the second half of the deck is poured and then a third phase, "closure pour" is used to connect the first two poured slabs. This closure pour can significantly extend the construction time and increase the cost of the deck.

The enclosed investigation assesses the deflection of a phased constructed steel girder bridge in Bellevue, Nebraska. The camber and deflection data, of the phases, from the design specs was compared to a numerical model and tilt sensor readings. The finite element model was analyzed the CSI Bridge structural analysis software. The numerical results were on trend with the design specs, although the values were slightly larger. Therefore, calibration is needed for the finite element model. Six weeks of daily deflection data was captured by the EL-tilt sensor. However, due to several issues the further data mining is required before comparisons can be made to the design plans and numerical results.

Acknowledgements

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The opinions expressed in this report are those of the authors and do not necessarily represent the opinions of the sponsors.

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Chapter 1. Introduction

Phased Construction, including a closure pour, is a common method used by State DOTs for bridge replacement. It allows for the traffic flow to be operated on half of the bridge while re-decking the other half. A number of problems might occur during bridge replacement and closure pour process. One of these problems involve the deck deflection. Dead load deflection or differential elevation greater than 2 inches prevents the second phase from being poured in one step, thus justifying the need for a closure pour. The added closure pour increases and extends the construction cost and time.

1.1 Objective

The main goal of this project is to investigate the issue of differential elevation between phases of a steel girder bridge, under construction. In pursuit of this goal, the following research objectives were proposed:

- Investigate the cause of differential elevation during the phased construction of steel girder bridges.
- Use numerical assessment to evaluate possible mitigation measures and differential elevation reduction techniques.
- Use sensed deflection data to compare with and calibrate the numerical model.

1.2 Scope

Based on the project objectives this report is organized into several chapters, which provide both background information and a description of the methodology.

Chapter 2 provides an overview of literature related to design considerations for phased construction, load considerations and issues with deflection. The survey instrument and its results are provided in Chapter 3. The numerical bridge model is illustrated in Chapter 4. The Plattsmouth Bridge information, schedule, and construction observation is discussed in Chapter 5. Comparison of results from the CSI Bridge model and collected data from sensors is provided in Chapter 6. Concluding remarks are presented in Chapter 7.

Chapter 2. Literature Review

2.1 Design Considerations for Phase Construction

In the AASHTO/NSBA (2010) document, design quality and value, fabrications, and construction of steel bridge, which might effect on phased construction and closure pour, were considered. Construction loading, which can control stress or deflections in structural behavior, need to be recognized and understood. The results and consequences of Cumulative loading effects in locked-in superstructure stresses might be a result by the sequence of construction. Permanent dead load deflection, transient live load deflection, stability of the partial and completed structure, and cross frame/diaphragm detailing would be several affects which should be considered (AASHTO 2010).

The same sets of section properties apply in phased construction method for steel girder bridges with the various loading conditions. Some girders might have a reduced composite section, which are in a given construction stage, regarding to the proximity of a longitudinal construction joint in the deck (stage line) momentarily. The designer must account for these differences in section properties (and loads) when evaluating strength and serviceability and also, maybe most importantly, must account for these differences when estimating girder deflections/cambers (AASHTO 2010). AASHTO (2003) recommends the use of a minimum of three girders to insure lateral stability of steel girders bridges.

Plattsmouth Bridge as a model used in this report has curved girders; therefore, load shifting would apply in this bridge. Torsion occurs in curved girders because the center of loading (center of gravity) is offset from the chord line somewhere in the middle of their supports. Load shifting effect causes that loads, which are carried by girders on the inside are different than those on the outside. Figure 2.1 shows load shifting behavior.

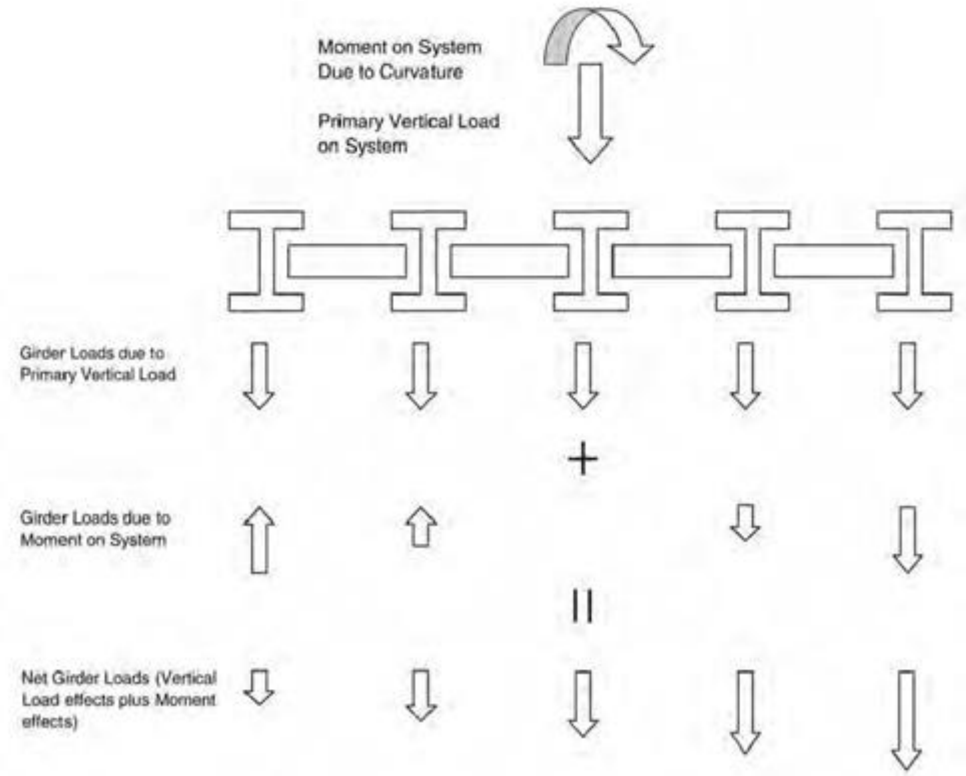

Figure 2.1 Illustration of Load – Shifting phenomenon experienced by curved girder

Dead (permanent) load deflection has effect on phased construction in deferential elevation of phases before and during closure pour. Section 3.3.2 in AASHTO/NSBA, illustrates weight of deck forming system's role in third phase of bridge construction. Slab weight, which can be applied as a uniformly distributed line and simple load on each girder, assumed that is applied to the non-composite, structural steel framing system. This section illustrated that type of forming system would affect dead load effect on phase construction. The type of forming system includes permanent or stay-in-place forms, and removable forms. Permanent forms will affect the nature of the effects of dead load in long term. When permanent forming is used, typically the effects of its weight are approximated using a simplified calculation or based on an approximate percentage of the weight of the deck; many owner agencies have different recommendations for this calculation which should be followed (AASHTO 2010).

Carefully consideration of dead load deflections is recommended while superstructure, which have a longitudinal joints in the deck slab. The girder camber diagram, deck haunch, and cross frame would be impacted by permanent load deflections.

The analysis of a structure with a longitudinal construction joint in the deck slab should first consider the number and sequence of transverse deck placements. Typically, the use of two transverse deck placements is the most common approach if partial phased construction is required. However, there may be benefits in providing a third transverse deck placement as a closure placement to minimize the impacts of differential deflections (AASHTO 2010).

The loads informed by temporary barriers can affect the determination of girder deflections depending on width of stage, spacing between girders, dimensions of temporary deck overhang, and any other factors which effect on structural behavior. In addition the eccentric load to the center of gravity of the stage bridge section cannot be ignored and should be considered in final deflections of girders.

The transient live load deflections effect on phase construction. Deflections and vibrations caused by live load impact the cross frame/diaphragm and the quality of the bridge slab finish while the deck placement and curing process. Allowing live loads on the structure during deck slab placement and curing can result in an uneven finish and cracking. However, when live loads must be maintained on a structure, the engineer should consider the design implications of this loading condition (AASHTO 2010). In order to ensure that superstructure design is not controlled by the temporary construction condition, understanding and studying the phase construction live load lane positions and temporary barriers would be essential. The composite girders with the deck slab support live loads in case of straight girder system completely; however, for typical horizontally curved girder systems, because of load shedding, the live load may effect in the portion of the superstructure which is not yet composite with the slab.

Consideration should be given to the cross frame/diaphragm detailing with respect to live load deflections. The connection detailing will determine whether girders adjacent to the composite superstructure will contribute to supporting the live loads and temporary barriers used during phased construction (AASHTO 2010).

2.2 Load Requirements

The purpose of "Bridge Office Policies and Procedures" (2013) manual is provide the standard and regulation, limitation, and guidance for bridge designers to use as a reference for preparation of plans and specifications for bridge to be constructed in Nebraska. Girder design policy in section 3.1.2 of BOPP-2013 illustrates deflection limits used to control deflection and compare with maximum live load deflection from our numerical model.

- Vehicular load, general span/800
- Vehicular and/or pedestrian loads span/1000

The Plattsmouth bridge spans' lengths are (span1=130', span2=193', and span3=139'). Because span2 is longest span, it is considered for maximum live load deflection. Maximum live load deflection in center of span 2 of phase 1 is shown in Table 2.1.

Table 2.1 Preferred Maximum Live Load Deflection in Center-Span 2 of Phase 1 (in)

PHASE 1 (Span 2)			
Girder	(Span 2) L (ft)	L/800	L/1000
A	191'-4 5/16"	2.870	2.296
B	191'-8 3/16"	2.875	2.300
C	192'-0 1/16"	2.880	2.304
D	192'-3 15/16"	2.885	2.308
E	192'-7 13/16"	2.890	2.312

"For all bridges on the State highway system, the load factor for vehicular live load (LL) and vehicular dynamic load allowance (IM) for Strength I in Table 3.4.1-1, Load Combination and Load Factors, of the AASHTO LRFD Bridge Design Specifications shall be increased from 1.75 to 2.0" (NDOR 2013).

2.3 Differential Elevation and other

Azizinamini et al. (2003) assessed the Dodge Street replacement bridge over I-480 in Omaha, NE. The bridge was monitored during and after construction for observation of problems. The observed problems were categorized into two groups, short term and long term. Short term concerns are related to the constructability issues while the long term concerns referred to the structural performance after the construction. Differential elevation at time of closure was considered a short term concern. Figure 2.2 illustrates differential elevation. From the study it was determined that potential causes of differential elevation between phases may include: construction error or tolerances, timing of the approach slab pour, creep and shrinkage and placement of temporary and permanent barriers. Creep and shrinkage can cause additional deflection after the closure, see Figure 2.3. The recommended remediation techniques involve the size and placements of temporary ballasts, temporary supports and inter-phase jacking (Azizinamini 2003).

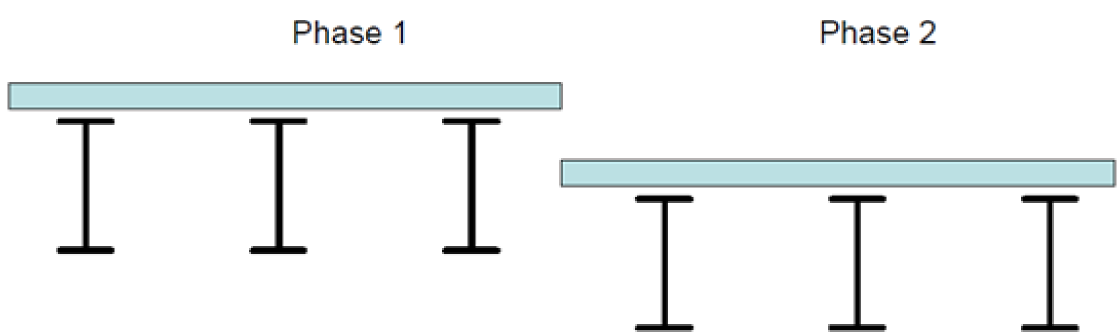

Figure 2.2 Example of differential elevation between phases (Azizinamini 2003)

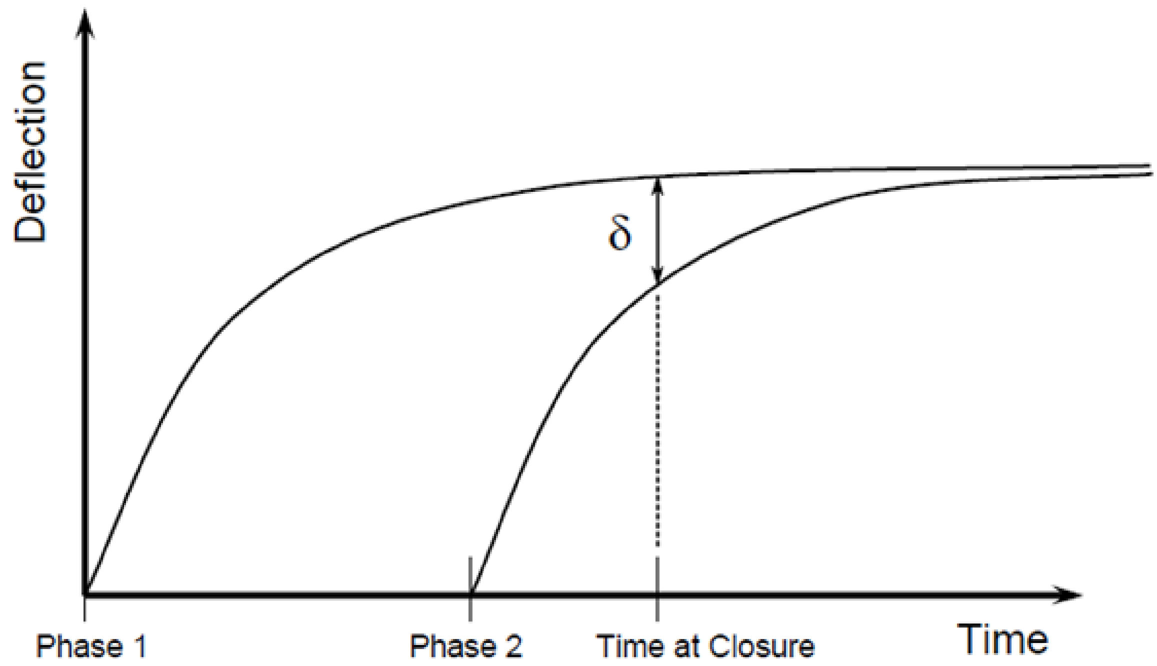

Figure 2.3 Creep and Shrinkage over Time

The 2004 project for the Wood River Bridge had issues with the camber of the second phase, causing problems with the deck thickness and steel clearance (NDOR 2004). T. Retterer (2012) analysis suggested lowering the bearing seat elevations of later phases to account for potential of higher camber.

The I-80 Hershey Interchange project (2008) was built in two phases with a 5 ft. closure pour. It was observed that the second phase did not deflect as much as the first. The workers attempted to resolve the problem by lining the deck with barriers and then stacking them at the mid-span, see Figure 2.4. Because this was unsuccessful, the closure pour was completed with mismatched deck elevations (NDOR 2008).

Figure 2.4: I-80 Hershey Interchange project in North Platte, NE after stacking barriers at the mid-span of second phase.

Camp Creek Bridges (2011) saw problems with concrete slump down the 2% cross slope of the deck, causing wet concrete bulge at bottom side of the slope. Observed deck cracking was resealed using BASF Degadeck sealant. In addition, it was determined that twice as many cracks were in the closure pour section after cure.

2.4 Monitoring and Assessment

Yakel et. Al. (2005) reported on the monitoring of the phase constructed Dodge Street Bridge over I-480 in Omaha, Nebraska. Period of monitoring was both short-term data, during construction events, and long-term data, daily and seasonally from October 20, 1999 through May 23, 2005. It was observed that environment, traffic, and time are main elements affecting the bridge (Yakel et. Al. 2005).

Chapter 3. Survey Assessment

Differential elevation has been an issue in Nebraska that requires a closure pour to connect the two phases of construction of steel girder bridges. Therefore a survey was created as a means of information collection, to determine if other Departments of Transportation (DOTs) are having similar issues and how they handle the problem. The questionnaire consisted of sixteen questions addressing design, construction and serviceability procedures and practices. The survey instrument is provided in Appendix B.1. The responses were kept confidential and only reported by state/regional location.

The survey was distributed electronically to representative from all 50 State DOTs. A total of 25 surveys were completed for a response rate of 50%. Figure 3.1 presents the number of positive responses, characterized by geographical region. When asked about the use of a closure pour 16 responded positively ('yes'). Although this is mostly done in a case by case situation, responders confirmed that deflection was the determining factor. Comments from question #4 is provided in Figure 3.2.

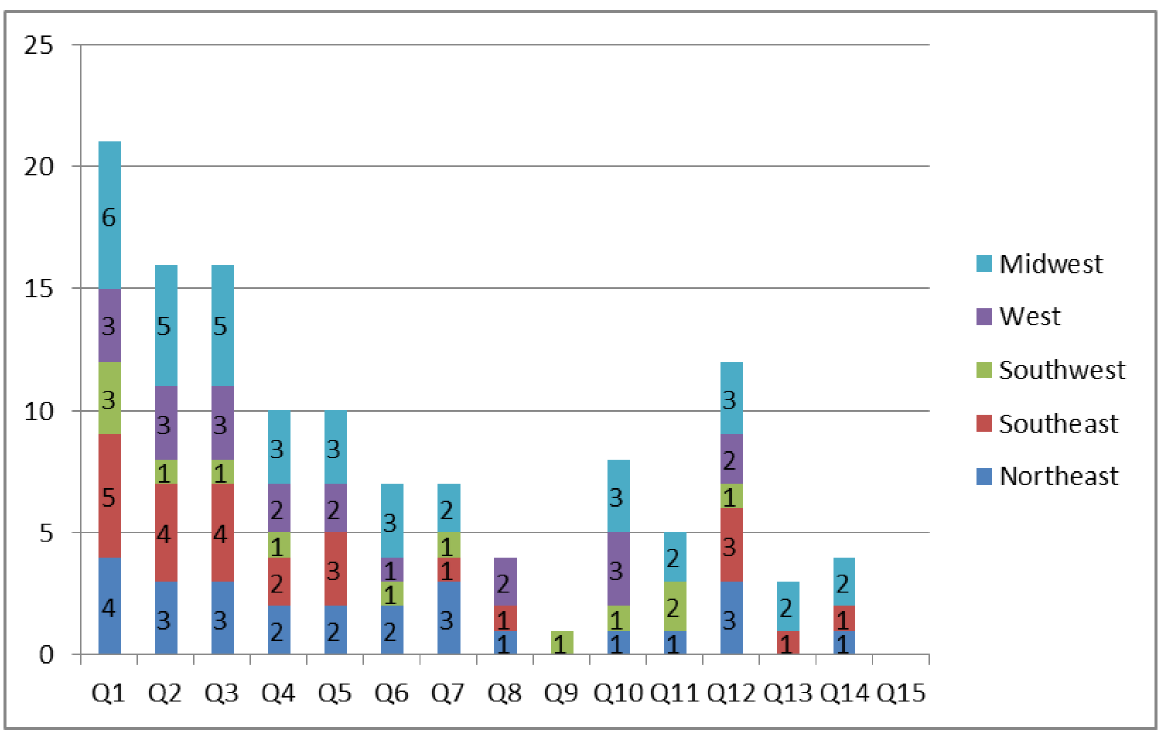

Figure 3.1. Positive Survey Responses

Q4. If yes, what justifies the need for a closure pour (i.e. dead load deflection exceeds 2 in)?

Region	Deflection	Other	No Answer
Northeast	2	2	0
Southeast	2	2	1
Southwest	1	1	2
West	2	1	0
Midwest	3	3	0

Region	Response
Northeast	“closure pour is prefered to reduce exposure to vibrations from adjacent stage 1 traffic.”
	“We typically have the longitudinal deck joint between the stages over a beam.”
Southeast	“Georgia uses closure pours only for continuous steel bridges that are constructed under traffic. For simple spans constructed under traffic, closure pours are not used.”
	“Required on steel girder bridges”
Southwest	“Phase construction issues are always taken on a case-by-case basis. Cross frames haven been temporarily left out, or they have been installed with slotted connection holes, all with varying degrees of success. Closure pours are employed when the deflecti...”
West	“We don't have a set criteria. It is a project by project discussion.”
Midwest	“A closure pour is considered at a longitudinal construction joint, on a case-by-case basis, if either of the following conditions applies.  1) The bridge deck will deflect more than 2 inches (50 mm) under dead load. 2) The staged bridge co...”
	“differential dead load deflection between phase construction exceeding 1/4".
	“Michigan typically does not require a longitudinal closure pour, however, we've been forced to on past deck replacement or superstructure replacements on curved and superelevated structures. Eliminating the parabolic curve in the deck, without changing t...”

Figure 3.2. Responses to Question #4, justification for closure pour

Chapter 4. Finite Element Model

4.1 Numerical Model Introduction

The numerical model of a steel girder bridge was developed in the CSI Bridge® Advanced software for finite element analysis (CSI 2011). This software has the capabilities to model, analyze and design bridge structures. It allows structural engineers to easily define complex bridge geometries, boundary conditions and load cases. The integrated SAPFire® analysis engine includes: staged construction, creep and shrinkage analysis, camber and shape finding, geometric nonlinearity (P-delta and large displacements), material nonlinearity (superstructure, bearings, substructure and soil supports), buckling and static and dynamic analysis.

All of these apply to a single comprehensive model. In addition, AASHTO LRFD design is included with automated load combinations, superstructure design and the latest seismic design" (CSI 2011).

4.2 CSI Bridge Modeling

Two different bridges were modeled in CSI Bridge in order to better understand the software's ability, integrity, and versatility for the project. These bridges were both designed and constructed in the state of Nebraska.

4.2.1 Hershey Interchange Bridge

Hershey Interchange Bridge is located in Lincoln County on I-80 interstate. The numerical model was developed in accordance with the bridge plan and the NDOR policies and procedures (BOPP 2013). The finite element model is shown in Figure 4.1 and its analyzed deformed shape for dead load is presented in Figure 4.2. The assumptions used and scaled dimensions used are the following:

- Abutment width 4ft (according to BOPP – Manual 2013, the limit of abutment wideness is not less of 3.5ft).

- Abutment depth 8ft.
- Elevation of top of abutment (-1) with angle of (3°11').
- Bearing elevation (-0.5) with no rotation on end of line.
- Distance from bottom of slab to existing ground is 18.32ft.
- Height of bridge from I-80 pavement is 16'11".
- Column is circle with 5ft.
- Columns distance from edge: (first column 9ft; second column 22.73ft; third column 36.40ft)
- Column height 21.10ft
- Bent cap depth 10ft.
- Bent cap width 5ft.

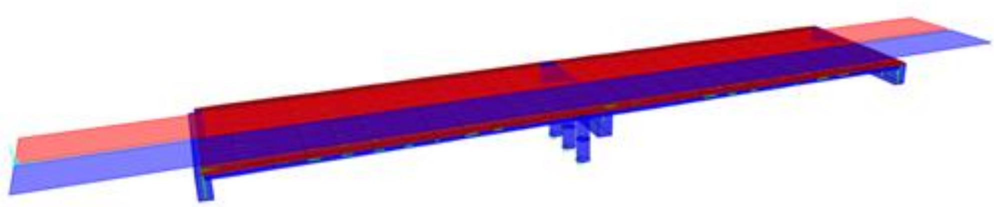

Figure 4.1 Numerical Model of Hershey St Interchange in North Platte, NE.

Figure 4.2 Hershey Interchange Deformed Shape under Dead Load

Table 4.1 presents the maximum displacement, in inches, of the steel girders to dead loads in both spans of the Hershey Bridge. Span 1 is greater than span 2.

Table 4.1: Maximum deformation of Girders (Hershey Bridge).

Max. Deflection of DL of Hershey Interchange Bridge (in)							
	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7
Span 1	6.3684	6.3864	6.414	6.418	6.389	6.335	6.287
Span 2	6.3204	6.3552	6.396	6.418	6.389	6.353	6.322

4.2.2 Plattsmouth Bridge Total Model

US75 Plattsmouth Bridge is located in Bellevue, NE. More details about the bridge site is provided in the next chapter. CSI Bridge 3D model of Plattsmouth Bridge is indicated in Figure 4.3. The numerical model was constructed in accordance with the bridge plans. The total bridge was model, including the closure pour section. Phase 1 is highlighted in red, while Phase 2 is highlighted in green. The software does not allow for the two phases to be modeled together without the closure pour section. For this, the phases were modeled individually and presented in the next section.

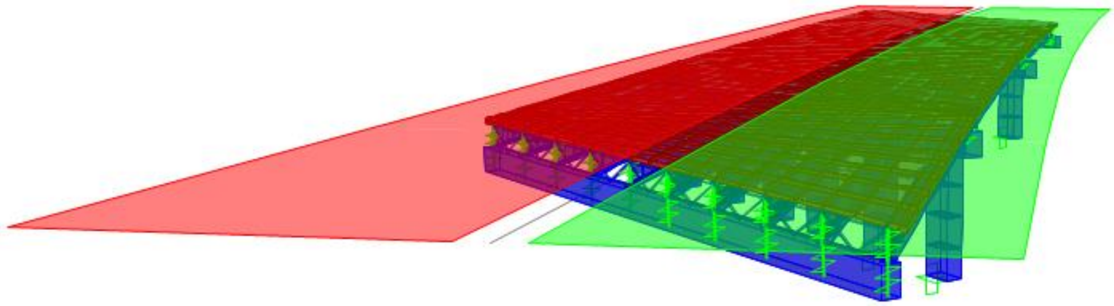

Figure 4.3 Completed 3D Model of Plattsmouth Bridge

The deflected shape of the entire bridge is given in Figure 4.4. Maximum displacements due to entire dead load (girders, slab, and concrete railing) are shown in Table 4.2.

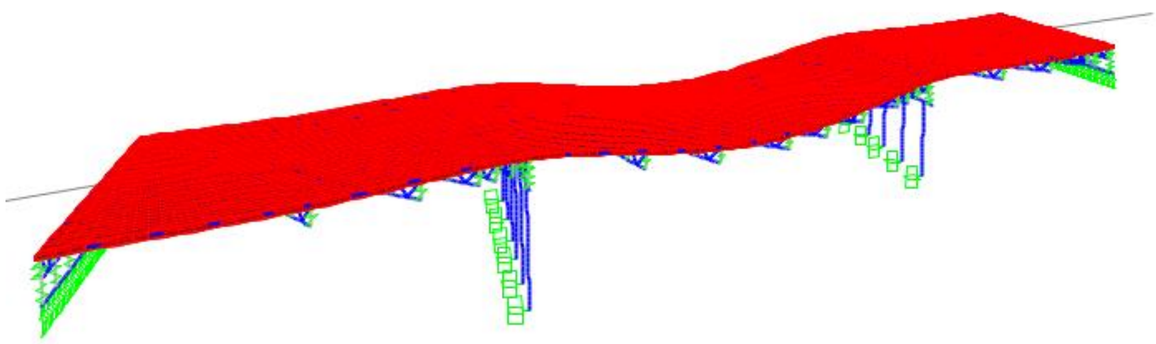

Figure 4.4 Dead load deflection of the Plattsmouth Bridge

Table 4.2 Maximum dead load (DL) deflection of all girders (Plattsmouth Bridge).

Max. Deflection of DL of Plattsmouth Bridge (in)					
PHASE 2					
	Girder A	Girder B	Girder C	Girder D	Girder E
Span 1	0.47	0.52	0.52	0.52	0.53
Span 2	8.74	8.18	7.71	7.40	7.23
Span 3	1.03	0.98	0.91	0.88	0.83
PHASE 1					
	Girder F	Girder G	Girder H	Girder I	Girder J
Span 1	0.56	0.58	0.61	0.64	0.65
Span 2	7.38	7.48	7.72	8.13	8.70
Span 3	0.80	0.80	0.79	0.79	0.76

4.2.3 Plattsmouth Bridge Phase Model (Individual Phase)

A finite element model of the individual phases was also developed, Phase 1 presented herein. The phased model is necessary for comparison with the deflections provided in the bridge plans and measurements collected from the EL tilt sensors. The single phase model and its deformation are provided in Figures 4.5, 4.6 and 4.7.

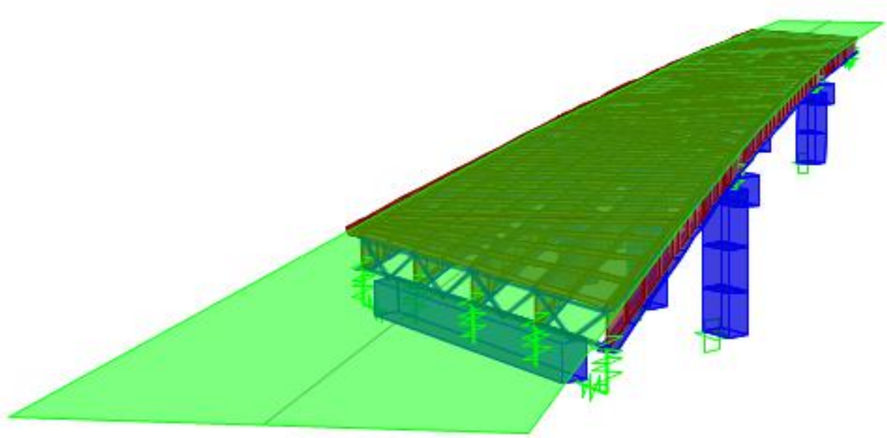

Figure 4.5: Completed 3D model of just one phase of the Plattsmouth Bridge.

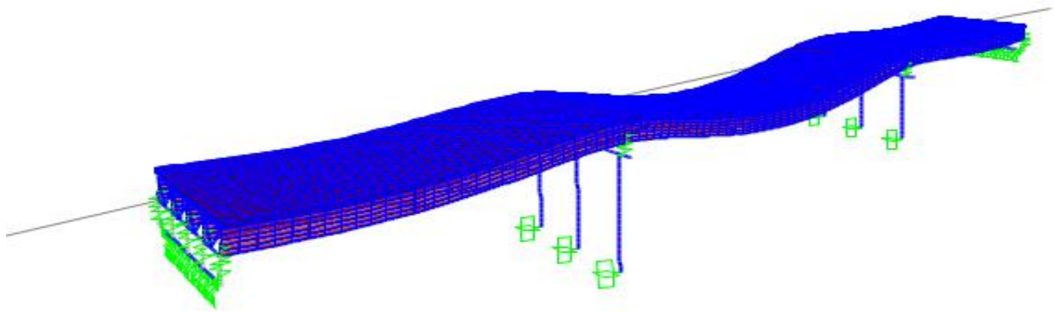

Figure 4.6: Dead load deflection of a single phase.

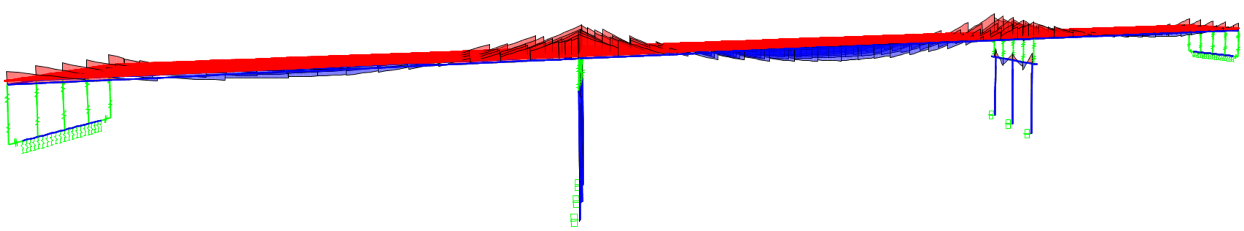

Figure 4.7 Dead load moment diagram of a single phase

The model of one phase has five girders including 3 interior and 2 exterior girders. It is essentially a stand-alone bridge, without a closure pour. Therefore, the load of closure pour was not being applied in this model. This simulates the first phase of the actual bridge on the Plattsmouth site. Thus, the deflection of girders in the model of entire bridge should be different than the displacement in the model of one phase. Tables 4.3, 4.4, and 4.5 provide the displacement comparison of the entire Plattsmouth Bridge model to a single phase (Phase 1) for Span1, Span 2, and Span 3, respectively. It is observed that the girder displacement results of the single phase model are greater than entire bridge model. This difference may be due to the difference in load distribution of the two models. In addition, because of software limitations the entire bridge model has one bent cap connecting the two phases instead of a separate bent cap for each phase.

Table 4.3: Maximum deflection of DL (Dead Load) of Span 1 of Phase 1 vs. entire Plattsmouth Bridge from CSI Bridge.

Max. Def. of DL in Span 1 of Phase 1 vs. Entire Plattsmouth Bridge (in.)					
	Girder F	Girder G	Girder H	Girder I	Girder J
Deflection (Phase 1)	1.267	1.362	1.476	1.627	1.841
Deflection (Entire Plattsmouth Bridge)	0.557	0.576	0.607	0.637	0.649
Difference in inches	**0.710**	**0.786**	**0.869**	**0.990**	**1.192**
Difference in Percentage	**56.1%**	**57.7%**	**58.9%**	**60.8%**	**64.7%**

Table 4.4: Maximum deflection of DL in Span 2 of Phase 1 vs. entire Plattsmouth Bridge from CSI Bridge.

Max. Def. of DL in Span 2 of Phase 1 vs. Entire Plattsmouth Bridge (in.)					
	Girder F	Girder G	Girder H	Girder I	Girder J
Deflection (Phase 1)	9.090	9.130	9.367	9.802	10.441
Deflection (Entire Plattsmouth Bridge)	7.380	7.482	7.723	8.134	8.701
Difference in inches	**1.710**	**1.648**	**1.644**	**1.668**	**1.740**
Difference in Percentage	**18.8%**	**18.0%**	**17.6%**	**17.0%**	**16.7%**

Table 4.5: Maximum deflection of DL of Span 3 of Phase 1 vs. entire Plattsmouth Bridge from CSI Bridge.

Max. Def. of DL in Span 3 of Phase 1 vs. Entire Plattsmouth Bridge (in.)					
	Girder F	Girder G	Girder H	Girder I	Girder J
Deflection (Phase 1)	2.557	2.364	2.195	2.124	2.162
Deflection (Entire Plattsmouth Bridge)	0.802	0.804	0.784	0.787	0.764
Difference in inches	**1.756**	**1.560**	**1.411**	**1.337**	**1.398**
Difference in Percentage	**68.7%**	**66.0%**	**64.3%**	**62.9%**	**64.7%**

4.3 Girder Tables

This section provided the girder lengths used to model the first phase in CSI Bridge. These lengths were estimated from the girder layout and elevation provided in the bridge plans. It should be noted that the girders are spliced together and vary in flange and web sizes. An example of this is provided in Figure 4.8.

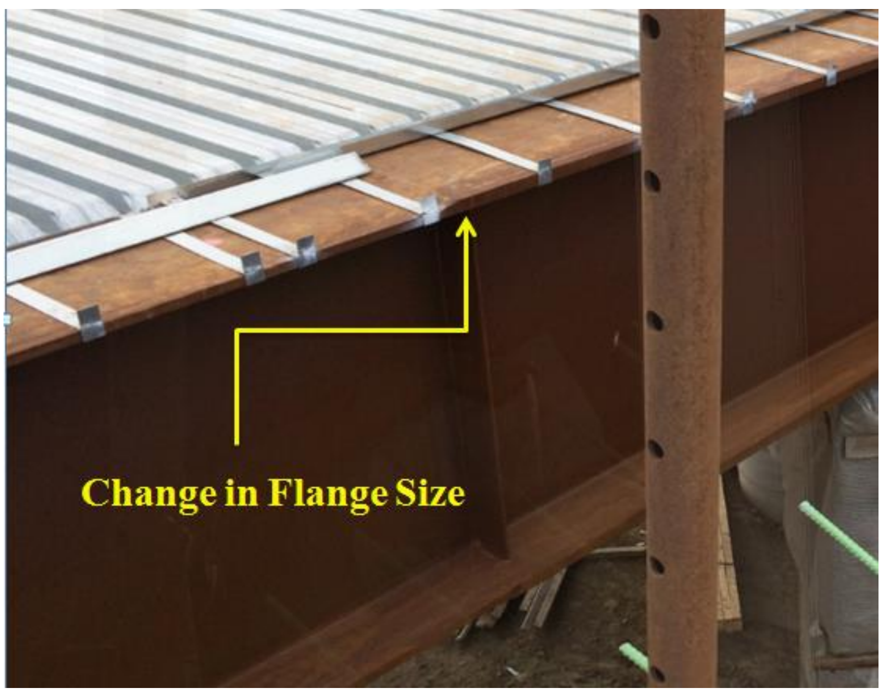

Figure 4.8: Plattsmouth Bridge Girder (February 28, 2014).

The girder data for each section of Phase 1 is provided below. Tables 4.6, 4.7, 4.8 and 4.9 indicate section length, type (size), and the label used in the CSI Bridge program. The total length, in units of feet, of the girders for all three spans is included in Table 4.9 and 4.10.

Table 4.6: Plattsmouth Bridge girder length in feet (Phase 1 - Span 1).

Group	Label	Label	Label	Label	Label	Label	Label	Label	Label	Label	TOTAL LENGTH
	Type	Type	Type	Type	Type	Type	Type	Type	Type	Type	
	Length	Length	Length	Length	Length	Length	Length	Length	Length	Length	
J	362	343	329	319	314	626	625	304	299	294	
	1	1	1	1	1	1	2	2	2	3	
	12.87	21.95	18.33	18.33	9.16	10.36	4.32	14.69	10.13	10.15	130.29
I	361	342	328	318	313	308	303	298	293		
	1	1	1	1	1	2	2	2	3		
	21.94	21.97	18.36	18.35	9.18	11.94	11.94	8.23	8.24		130.15
H	360	341	327	317	312	307	386	385	292		
	1	1	1	1	2	2	2	3	3		
	31.00	22.00	18.38	18.38	9.19	9.19	10.86	4.67	6.33		130.00
G	369	359	340	326	388	387	311	390	389	291	
	1	1	1	1	1	2	2	2	3	3	
	21.51	18.53	22.03	18.41	10.53	7.88	9.21	10.77	2.12	8.88	129.86
F	373	368	358	339	325	315	310	290			
	1	1	1	1	1	2	2	3			
	13.75	13.77	21.55	22.05	18.43	18.44	9.22	12.50			129.72

Table 4.7: Plattsmouth Bridge girder length in feet (Phase 1 - Span2).

Group	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	TOTAL LENGTH	TOTAL from last TABLE
J	275	4	3	265	260	255	241	226	212	198		
	3	3	2	2	2	4	4	4	4	4		
	8.20	**3.80**	**5.17**	**8.97**	**8.97**	**8.97**	**14.54**	**16.45**	**14.62**	**16.90**	**106.59**	**236.88**
I	284	10	9	269	264	259	254	240	225	211		
	3	3	2	2	2	4	4	4	4	4		
	5.87	**6.54**	**5.19**	**8.99**	**8.99**	**8.99**	**8.99**	**14.48**	**16.47**	**14.64**	**99.13**	**229.28**
H	283	12	11	273	268	263	258	253	239	224		
	3	3	2	2	2	4	4	4	4	4		
	9.00	**3.00**	**6.00**	**9.00**	**9.00**	**9.00**	**9.00**	**9.00**	**14.50**	**16.50**	**94.00**	**224.00**
G	282	277	272	267	262	257	252	238	223	209		
	3	2	2	4	4	4	4	4	4	4		
	12.11	**12.13**	**12.13**	**9.01**	**9.01**	**9.01**	**9.01**	**14.52**	**16.53**	**14.69**	**118.17**	**248.03**
F	281	276	271	266	261	256	251	237	222	208		
	3	2	2	4	4	4	4	4	4	4		
	15.22	**15.25**	**15.27**	**9.03**	**9.03**	**9.03**	**9.03**	**14.55**	**16.55**	**14.72**	**127.68**	**257.39**

Table 4.8: Plattsmouth Bridge girder length in feet (Phase 1 - Span2) (cont'd).

Group	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	TOTAL LENGTH	TOTAL from last TABLE
J	193	188	183	178	6	5	168	8	7			
	4	4	4	4	4	5	5	5	5			
	9.59	**9.59**	**9.59**	**9.59**	**8.75**	**7.46**	**16.22**	**5.26**	**11.00**		**87.06**	**323.94**
I	197	192	187	182	177	172	167	162				
	4	4	4	4	4	5	5	6				
	16.93	**9.61**	**9.61**	**9.61**	**9.61**	**12.92**	**12.92**	**12.94**			**94.15**	**323.43**
H	210	196	191	184	181	176	171	166	161			
	4	4	4	4	4	5	5	5	6			
	14.67	**16.96**	**9.63**	**9.63**	**9.63**	**9.63**	**9.63**	**9.63**	**9.63**		**99.00**	**323.00**
G	195	190	185	180	175	170	160					
	4	4	4	5	5	5	6					
	16.99	**9.64**	**9.64**	**9.64**	**9.64**	**6.34**	**12.67**				**74.55**	**322.58**
F	194	189	184	179	174	159						
	4	4	5	5	5	6						
	17.01	**9.66**	**9.66**	**9.66**	**9.66**	**9.13**					**64.78**	**322.17**

Table 4.9: Plattsmouth Bridge girder length in feet (Phase 1 - Span 3).

Group	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	Label Type Length	TOTAL LENGTH	TOTAL from last TABLE
J	139	129	14	13	105	86	76	61				
	6	5	5	7	7	7	7	7				
	14.48	**17.20**	**4.38**	**12.83**	**15.30**	**20.96**	**27.07**	**27.11**			**139.32**	**463.26**
I	153	16	15	133	123	109	90	75	60			
	6	6	5	5	7	7	7	7	7			
	5.29	**6.71**	**3.88**	**17.22**	**17.23**	**16.27**	**21.98**	**28.61**	**21.97**		**139.16**	**462.59**
H	18	17	137	127	117	103	84	59				
	6	5	5	7	7	7	7	7				
	12.00	**5.25**	**17.25**	**17.25**	**17.25**	**15.33**	**21.00**	**33.67**			**139.00**	**462.00**
G	151	146	141	136	126	116	102	83	2			
	6	5	5	7	7	7	7	7	7			
	11.93	**11.95**	**11.96**	**8.64**	**17.28**	**17.27**	**15.35**	**21.02**	**23.43**		**138.83**	**461.41**
F	150	145	20	19	135	125	115	96	1			
	6	5	5	7	7	7	7	7	7			
	15.24	**15.27**	**8.29**	**7.00**	**8.65**	**17.30**	**17.30**	**23.05**	**26.56**		**138.65**	**460.82**

Table 4.10: Comparison of length in plan and CSI Bridge.

Girders	Length of Girder in Plans (feet)	Length of Girder in CSI Bridge (ft)	Difference (ft)
F	461.95	460.82	1.13
G	462.72	461.41	1.31
H	463.50	462.00	1.50
I	464.27	462.59	1.68
J	465.05	463.26	1.79

The average length difference is 1.48 ft. This difference is due to the change in radius of curvature along girders of each span in plan. In the numerical model the radius of curvature is kept constant along the bridge length.

Chapter 5. Bridge Field Assessment

5.1 Site Visits

5.1.1 Plattsmouth Bridge Location

The phased constructed steel girder bridge considered in this project was one that was under construction. The bridge is US75 Plattsmouth-Bellevue in Cass County crosses over Union Pacific railroad tracks. An aerial map of the bridge site is shown in Figure 5.1.

Figure 5.1 Plattsmouth Bridge site from Google Map

5.1.2 Plattsmouth Bridge Information

The Plattsmouth Bridge is located along US75 in Bellevue; NE. This project's number is 75-2 (167); C.N.: 21849E; structure number: S034 38219; Station: 1375+45.00; REF. POST.: 382.19; HWY. No.: US 34; County: Cass. The information from the bridge plan is as follow:

- 3 spans (130'+193'+139'=462ft)
- Abutment (width=3.5ft and Depth=5ft)
- Top of abutment Elevation=-6 with skew=45º.
- Bearing Elevation=-5.5 with skew=45º.
- Rectangular shape Columns (Width=4.5ft and Depth= 8ft)

- Bent Cap (width=5ft, Depth=6ft, and Length=66ft for each group of Columns)
- 6 Columns for each Bent.
- Design live load (HL-93)
- Bent Elevation=0 with skew=45º.
- 8 interior girders; total 10 girders.
- Left and Right Exterior girders overhang Length=3.083ft
- Left and Right Ext. girders Overhang distance of fillet=0.75ft
- Slab Thickness=8" and Overhang Thickness=1ft
- Girder Spacing @ 14' 10"
- Girders: Flange Thickness, Flanges width, and Web thickness are vary along spans

5.1.3 Construction Schedule

The construction schedule for this project was the following:

<u>Phase 1:</u>

- 08/12 to 01/13, Drive H-Pile, Build Abutments and Piers.
- 01/13 to 05/13, Set Girders, Install Stay-in-Place Decking, Place Re-Steel and Pour Deck.
- 05/13 to 06/13, Build Concrete Bridge Rail and Pour Approach Slabs.

<u>Phase 2:</u>

- 11/13 to 01/14, Drive H-Pile, Build Abutments and Piers.
- 01/14 to 05/14, Set Girders, Install Stay-in-Place Decking, Place Re-Steel and Pour Deck.
- 05/14 to 06/14, Build Concrete Bridge Rail and Pour Approach Slabs.
- 06/14 to 07/14, Install the Stay-in-Place Decking, Place Re-Steel and Make Closure Deck Pour.

Site visits were made during the construction process and documented via photograph. Figure 5.2 through Figure 5.10 present the construction sequence.

Figure 5.2 Phase 1 – Concrete Cure Process (May 22, 2013)

Figure 5.3 Completed Phase 1 under traffic load (September 14, 2013)

Figure 5.4 Phase 1 under slab, girders, cross frames (diaphragms), bent cap, and columns

Figure 5.5 Phase 2 piers excavation

Figure 5.6 Setting of Phase 2 girders (January 24, 2014)

Figure 5.7 Preparation for Phase 2 deck pour (April 16, 2014)

Figure 5.8 Completed Phase 2 with preparation for closure pour (May 14, 2014)

Figure 5.9 Phase 3 installed closure pour/ center median (May 14, 2014)

Figure 5.10 Complete bridge (June 30, 2014)

5.2 Instrumentation and Monitoring

5.2.1 Sensor Introduction and Description

Deflection results of steel girders under slab load are monitored by sensors in order to compare with CSI Bridge modeling and architectural plan predicted results. Sensor package device used in this project consist of:

- The EL tilt sensors
- 3 feet beams
- The EL Nulling Device
- SC115 CS I/O 2G Flash Memory Drive
- Data logger; Campbell Scientific CR1000
- Connection cable between sensors and data logger.

The sensor package is provided by Durham Geo Slope Indicator. The EL tilt sensor used to monitoring changes in the disposition and deflection of a structure is a narrow-angle, high-resolution device.

Figure 5.11 and 5.12 indicate the horizontal and vertical EL tilt sensor respectively. Dimensions of enclosure are (4.9"x3.2"x2.3").

Figure 5.11 the Horizontal EL Tilt Sensor

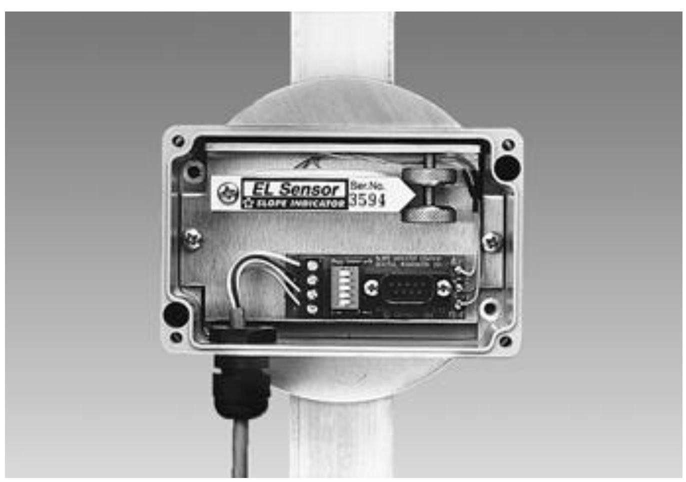

Figure 5.12 Vertical EL Tilt Sensor, interior

There are several applications for EL tilt sensor, including the following:

- Monitoring stabilization measures; for instance, grouting and underpinning pressure.
- Monitoring structures; such as effects of tunneling and excavating.
- Monitoring effects of load on structures.
- Monitoring behavior of retaining walls as far as deflection and deformation under load.
- Monitoring the rotation of piers, retaining walls, and piles.
- Monitoring tunnels' movement and convergence.

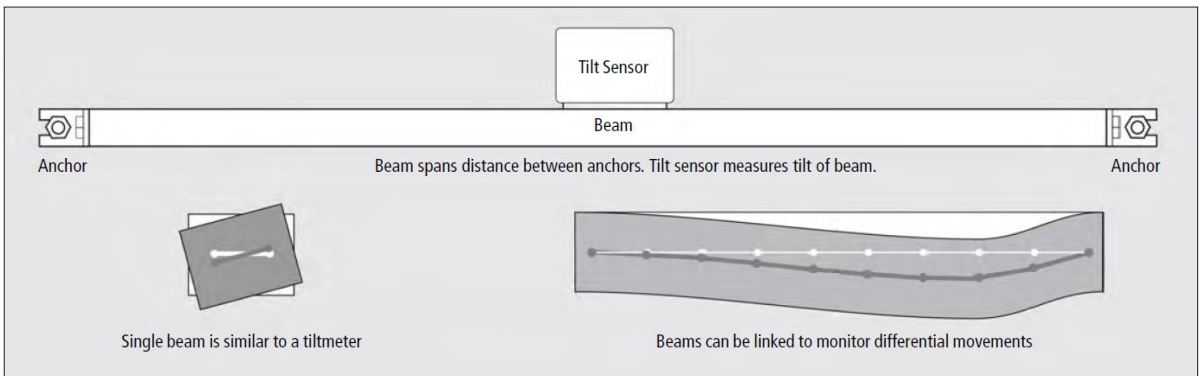

Figure 5.13 Horizontal Beam Sensors

The EL tilt sensor is an electrolytic tilt sensor held in a small, weatherproof enclosure. As shown in Figure 5.13 the EL tilt sensor can be installed on beam or tilt meter. In order to monitoring differential movement beam sensors are often connected in arrays. The EL tilt sensor compared to other sensors has several advantages:

- High Resolution: one second of arc would be the EL tilt sensor change detection in tilt.
- Robust and Reliable: it is protected by a weatherproof enclosure with no moving parts.
- Easy to install: flexible install position of versatile brackets make quick and easy placement for the sensors.

- Re-Configurable: The EL tilt sensor can be applied as tilt meter or/and beam sensor in different sites process.
- Cost Effective: its competitive price besides its advantages is considerable.

The EL tilt sensors used in the project are the standard version which works with the Campbell Scientific CR1000 data logger, see Figure 5.14. Range of sensor is ±40 arc minutes; resolution is 1 arc second using a Campbell Scientific CR1000 data logger with repeatability of ±3 arc second. (One arc second is 1/60 arc minute).

Before the mounting bracket is secured, the sensor can be zeroed, adjusted ±4º. The role of Omni bracket is to install and hold the tilt sensor onto inclined, horizontal, or vertical beam. For this project the EL tilt sensors are installed onto 3 feet long horizontal beam, which is clamped to the bridge girder. Operation temperature for EL tilt sensor is from -20ºC (-4ºF) to +50ºC (+122ºF). Data is sent to data logger CR1000 by shielded cable consists of four 24-gauge tinned-copper conductors covered with PVC jacket. Sensor measurements is extracted from the data logger using a laptop PC cable connection. Measurements are sensed every hour.

The CR1000 data logger is battery operated and it provides accurate measurement capabilities in a rugged condition. Some of its capabilities and features consist of 4 MB memory, program execution rate of up to 100 Hz, CS I/O and RS-232 serial ports, 13-bit analog to digital conversions, 16-bit H8S Renesas Microcontroller with 32-bit internal CPU architecture, and Battery-backed SRAM memory and clock ensuring data, programs, and accurate time are maintained while the CR1000 is disconnected from its main power source.

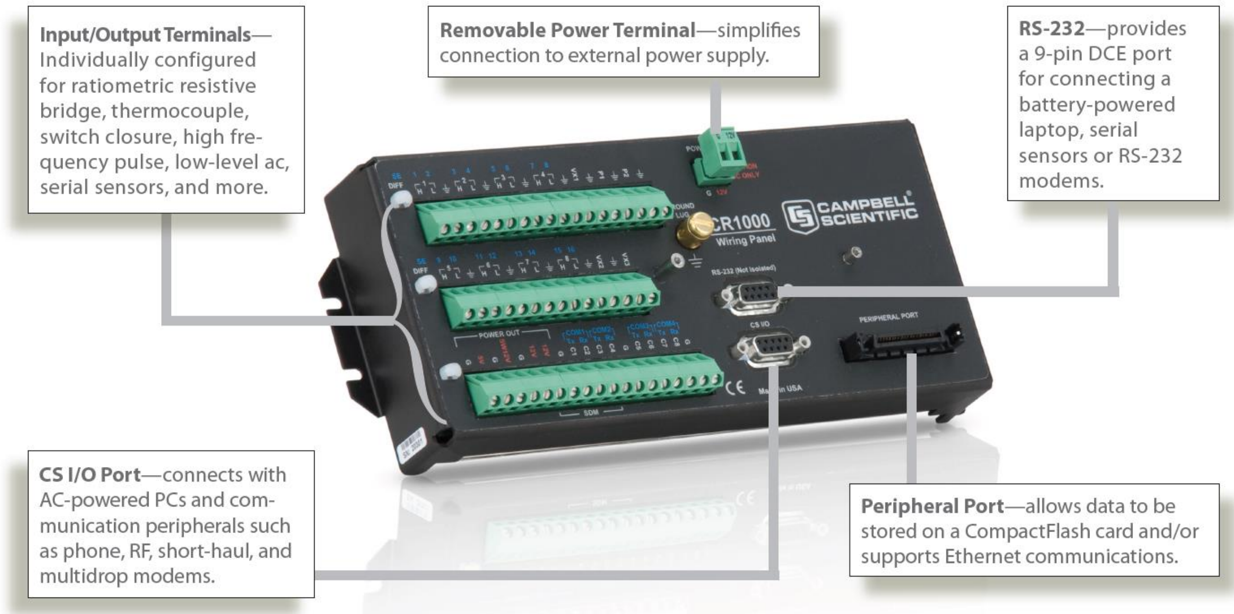

Figure 5.14 CR1000 Data logger

5.3 Sensor Measurement

5.3.1 Sensor Installation at Site

The horizontal EL tilt sensor beams were installed under Plattsmouth Bridge girders A – G of span 3, by C-clamps, shown in Figure 5.15.

Figure 5.15 EL tilt sensor beam installed under a girder at Plattsmouth Bridge

Seven sensors were allocated for five girders (all girders) of phase 2 – span 3 and two girders of phase 1 – span 3. Sensors' numbers and specific spots' dimensions are indicated in Table 5.1. and Figure 5.16 shows field splice at Plattsmouth Bridge's girder for reference.

Table 5.1 Girders, Sensor Numbers, and Install spots of sensors

		Distance from
Griders	**Sensor No.**	**Field Splice #4**
Phase2 A	17480	57'-7"
Phase2 B	17478	57'-8 15/16"
Phase2 C	17487	57'-10 1/2"
Phase2 D	17485	58'-1/8"
Phase2 E	17482	58'-1 11/16"
Phase1 F	17486	58'-2"
Phase1 G	17484	59'

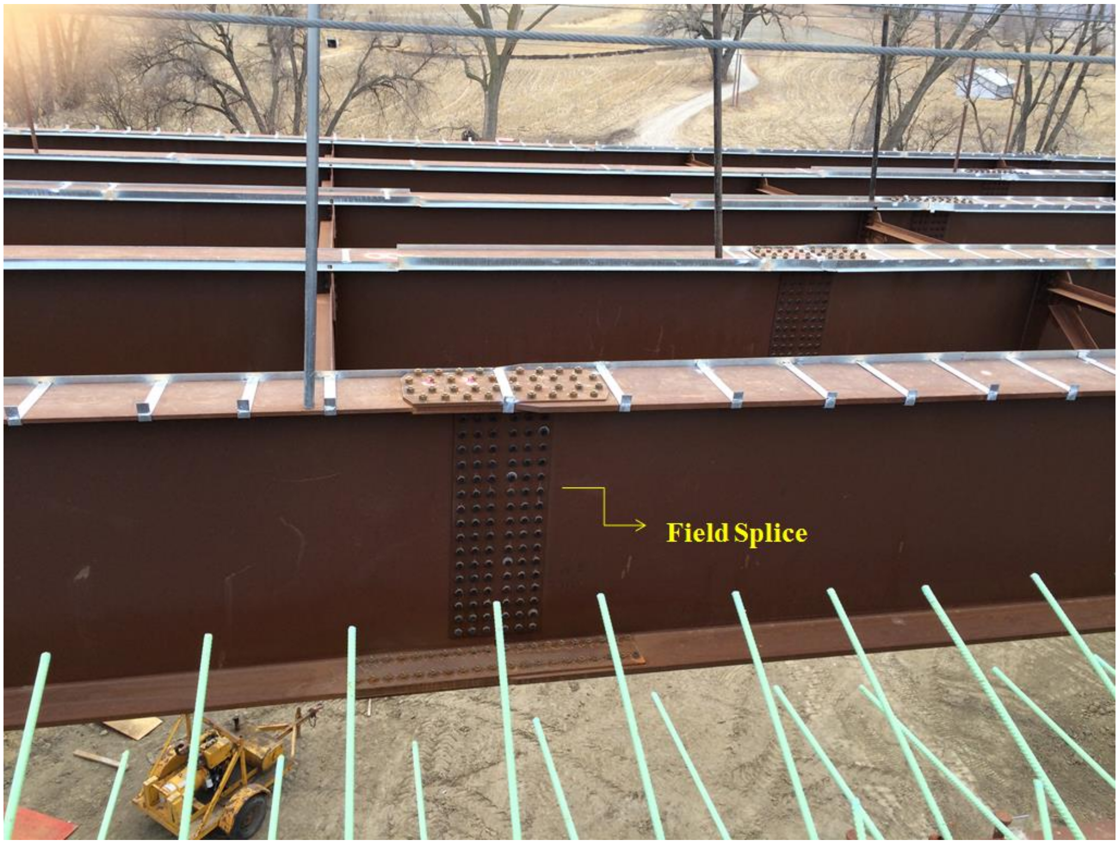

Figure 5.16 Girder field splice (February 28, 2014)

The data logger box was installed and connected to sensors by cables as show in Figure 5.17 and 5.18. The system was powered by a deep-cycle marine battery. The data logger consists of a CR1000 wiring panel, multiplexer, and PS100 power supply (connects to battery).

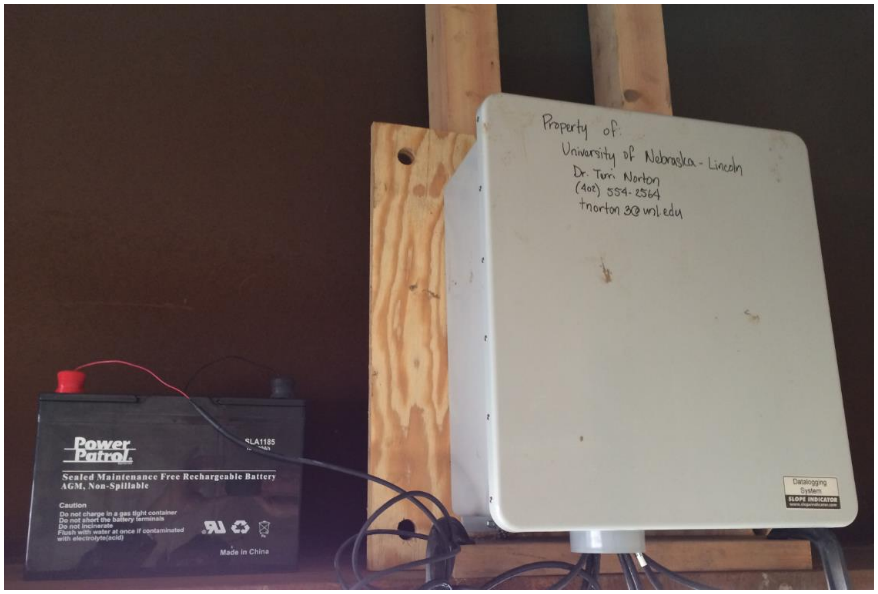

Figure 5.17 Data logger Box and Battery

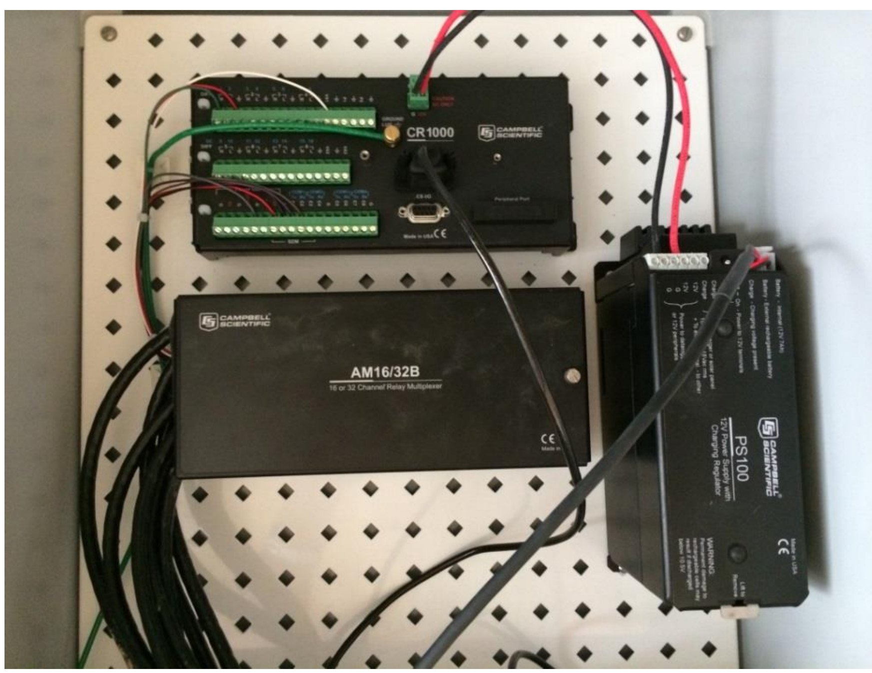

Figure 5.18 Inside Data logger Box

Chapter 6. Numerical Results and Comparison

6.1 CSI Bridge Results vs. HDR Consultant Plan (Plattsmouth Bridge)

In this chapter, obtained deflection results from CSI Bridge model for Plattsmouth Bridge are compared with HDR consultant plans. Load factor for dead load which consists of steel girder, slab, median, and concrete rail is 1.0.

Tables 6.1, 6.2, and 6.3 indicate comparison of maximum deflection for just steel girders in span 1, span 2, and span 3 of Phase 1 of Plattsmouth Bridge from CSI Bridge model with plan, respectively.

Table 6.1 Maximum Deformation of Girders in Span 1 vs. Steel Deflection in Plan

Max. Deformation Girders in Span 1 of Phase 1 vs. Steel Deflection from Plan (in.)					
	Girder F	Girder G	Girder H	Girder I	Girder J
Girder Deflection (CSI Bridge)	0.434	0.420	0.420	0.432	0.464
Steel Deflection (Span1 in Plan)	0.225	0.233	0.256	0.292	0.352
Difference of CSI results W/ Plan	**-0.209**	**-0.187**	**-0.164**	**-0.140**	**-0.112**
Difference in Percentage	**48.20%**	**44.52%**	**39.05%**	**32.41%**	**24.20%**

Table 6.2 Maximum Deformation of Girders of Span 2 vs. Steel Deflection in Plan

Max. Deformation Girders in Span 2 of Phase 1 vs. Steel Deflection from Plan (in.)					
	Girder F	Girder G	Girder H	Girder I	Girder J
Girder Deflection (CSI Bridge)	2.411	2.340	2.324	2.364	2.464
Steel Deflection (Span 2 in Plan)	1.651	1.610	1.611	1.657	1.749
Difference of CSI results W/ Plan	**-0.760**	**-0.730**	**-0.713**	**-0.707**	**-0.715**
Difference in Percentage	**31.52%**	**31.20%**	**30.69%**	**29.91%**	**29.01%**

Table 6.3 Maximum Deformation of Girders of Span 3 vs. Steel Deflection in Plan

Max. Deformation Girders in Span 3 of Phase 1 vs. Steel Deflection from Plan (in.)					
	Girder F	Girder G	Girder H	Girder I	Girder J
Girder Deflection (CSI Bridge)	0.752	0.689	0.659	0.656	0.686
Steel Deflection (Span 3 in Plan)	0.603	0.529	0.484	0.460	0.459
Difference of CSI results W/ Plan	**-0.149**	**-0.160**	**-0.175**	**-0.196**	**-0.227**
Difference in Percentage	**19.86%**	**23.20%**	**26.53%**	**29.92%**	**33.13%**

Maximum deflections of total dead load including girder, concrete slab, median, and concrete rail in span 1, span 2, and span 3 of Phase 1 are compared with DL deflection for shims (sum of slab deflection and super DL deflection) in plan. They are shown in Tables 6.4, 6.5, and 6.6.

Table 6.4 Max. Deflection of Dead Load of Span 1 vs. DL Deflection for Shims in Plan

Max. Deflection of Girder, Slab, and Conc. Rail in Span 1 of Phase 1 vs. DL Deflection for Shims (in.)					
	Girder F	Girder G	Girder H	Girder I	Girder J
Deflection (CSI Bridge)	0.557	0.576	0.607	0.637	0.649
DL Deflection for Shims (Span1 in Plan)	1.420	1.390	1.450	1.624	1.941
Difference of CSI results W/ Plan	**0.863**	**0.814**	**0.843**	**0.987**	**1.292**
Difference in Percentage	**60.77%**	**58.56%**	**58.14%**	**60.78%**	**66.56%**

Table 6.5 Max. Deflection of Dead Load of Span 2 vs. DL Deflection for Shims in Plan

Max. Deflection of Girder, Slab, and Conc. Rail in Span 2 of Phase 1 vs. DL Deflection for Shims (in.)					
	Girder F	Girder G	Girder H	Girder I	Girder J
Deflection (CSI Bridge)	7.380	7.482	7.723	8.134	8.701
DL Deflection for Shims (Span2 in Plan)	5.585	5.371	5.305	5.407	5.693
Difference of CSI results W/ Plan	**-1.795**	**-2.111**	**-2.418**	**-2.727**	**-3.008**
Difference in Percentage	**24.32%**	**28.21%**	**31.31%**	**33.53%**	**34.57%**

Table 6.6 Max. Deflection of Dead Load of Span 3 vs. DL Deflection for Shims in Plan

Max. Deflection of Girder, Slab, and Conc. Rail in Span 3 of Phase 1 vs. DL Deflection for Shims (in.)					
	Girder F	Girder G	Girder H	Girder I	Girder J
Deflection (CSI Bridge)	0.802	0.804	0.784	0.787	0.764
DL Deflection for Shims (Span3 in Plan)	2.633	2.292	2.079	1.992	2.054
Difference of CSI results W/ Plan	**1.831**	**1.488**	**1.295**	**1.205**	**1.290**
Difference in Percentage	**69.56%**	**64.92%**	**62.29%**	**60.48%**	**62.78%**

The numerical results shows larger deflection for span 2 when compared to the other spans. This is consistent with the deflection table of the bridge plans. However, the numerical deflections for the steel alone were considerably larger than that of the plans. This difference could be attributed to several things including: uncertainty of consultant's assessment constraints, degree of curvature, numerical assumptions and default inputs. The structural analysis program used by the design consultant is unknown. In addition, the numerical model may require calibration. Therefore sensors were installed on the bridge to monitor deflection.

6.2 Collected Data from Sensors

Collected data from sensors need to be calibrated by specific factors and equation. Each sensor has individual calibration factors, given in Table 6.7. The calibration factors are used to convert the voltage measurement to unit length (mm) per beam gauge length (m). The sensors were attached to 3 ft (0.9144m) beams. The Poly factors were used as they allow for a greater range of movement for the sensors.

Table 6.7 A, B Polynomial/Linear Calibrate Factors for Sensor No. 17480 at Girder A

A

Polynomial Factors

(Range of +/- 0.688 degrees)

C5	-0.01246
C4	0.3069
C3	-2.987663
C2	14.3574
C1	-34.31336
C0	33.21818

B

Linear Factors

(Range of +/- 0.1146 degrees)

m	-0.283674569
b	1.42097726

The deviation equation is provided in equations 6.1 below.

$$\text{Deviation (Poly Factors)} = C_5X^5 + C_4X^4 + C_3X^3 + C_2X^2 + C_1X + C_0 \text{ (mm/m)} \qquad 6.1$$

Deflection measurements were collected from May 14, 2014 to June 22, 2014, recorded each hour for 24 readings per day. The collected data for each girder can be found in Appendix C. In order to determine the daily deflection change for steel girders, the maximum reading data for each day (24 hours) was calculated. The sensors' maximum reading data per day, in volts, was converted to unit length using the polynomial factors and deviation equation presented above. The length of the gauge attachment beam of 3 feet is taken into account when converting the sensed data.

Table 6.8 provide a sample of data for Girder A. Column C, Change, gives the deflection difference of each day from the initial reading. The daily deflection for each girders in presented graphically in Figures 6.1 through 6.7.

Table 6.8: Daily deflection change of Girder A (Sensor No. 17480).

	A EL Reading	B Deviation Poly	C		A EL Reading	B Deviation Poly	C
Date	**Volts**	**in.**	**Change**	**Data**	**Volts**	**in.**	**Change**
23-May	4.55548	0.00487		8-Jun	4.543796	0.00499	0.00012
24-May	4.553751	0.00488	0.00002	9-Jun	4.547174	0.00495	0.00008
25-May	4.559354	0.00483	-0.00004	10-Jun	4.550863	0.00491	0.00005
26-May	4.548342	0.00494	0.00007	11-Jun	4.542952	0.00499	0.00013
27-May	4.54699	0.00495	0.00009	12-Jun	4.543428	0.00499	0.00012
28-May	4.55473	0.00487	0.00001	13-Jun	4.546132	0.00496	0.00010
29-May	4.561487	0.00481	-0.00006	14-Jun	4.523119	0.00520	0.00033
30-May	4.556757	0.00485	-0.00001	15-Jun	4.536669	0.00506	0.00019
31-May	4.537345	0.00505	0.00018	16-Jun	4.535459	0.00507	0.00020
1-Jun	4.540228	0.00502	0.00016	17-Jun	4.528734	0.00514	0.00027
2-Jun	4.545271	0.00497	0.00010	18-Jun	4.528555	0.00514	0.00027
3-Jun	4.538877	0.00504	0.00017	19-Jun	4.555406	0.00487	0.00000
4-Jun	4.539555	0.00503	0.00016	20-Jun	4.534281	0.00508	0.00022
5-Jun	4.546808	0.00495	0.00009	21-Jun	4.547298	0.00495	0.00008
6-Jun	4.545271	0.00497	0.00010	22-Jun	4.537362	0.00505	0.00018
7-Jun	4.538202	0.00504	0.00018				

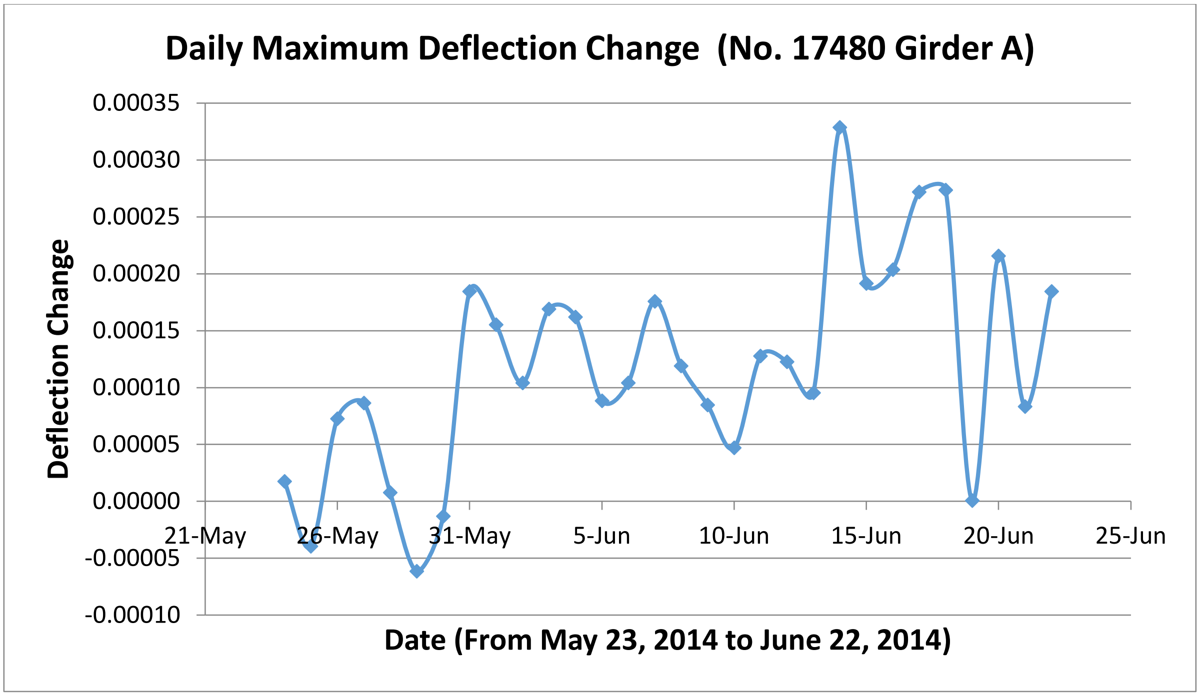

Figure 6.1: Daily maximum deflection change of Girder A.

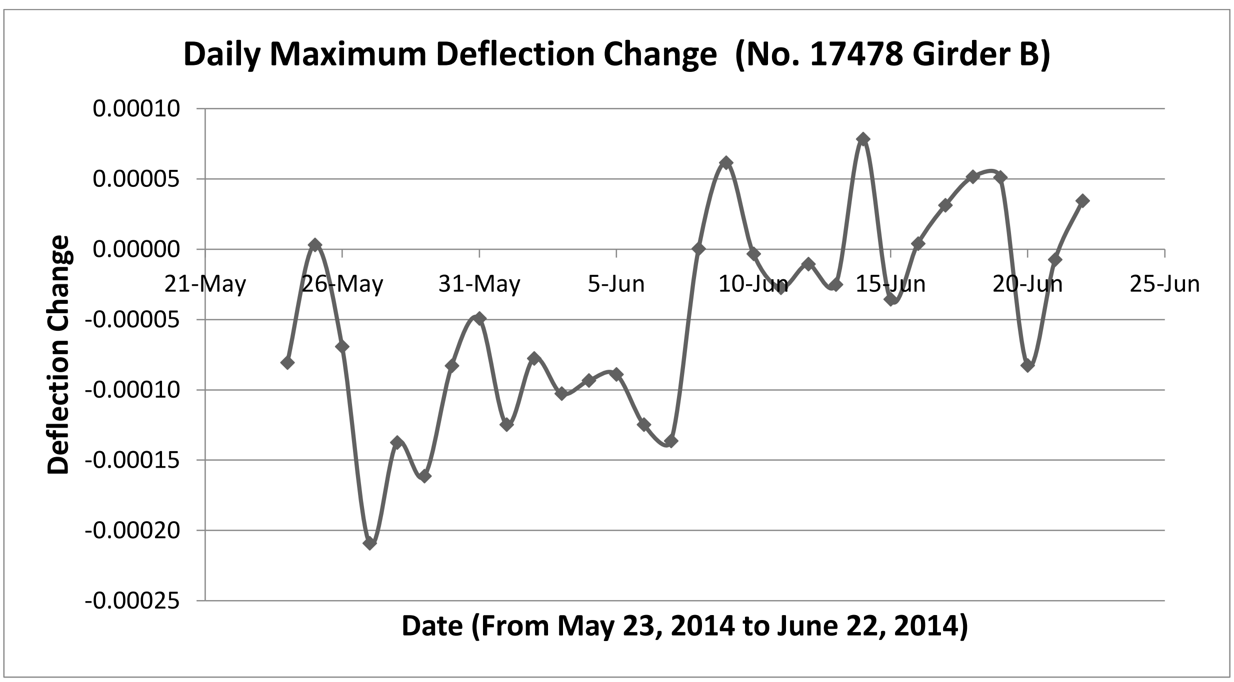

Figure 6.2: Daily maximum deflection change of Girder B.

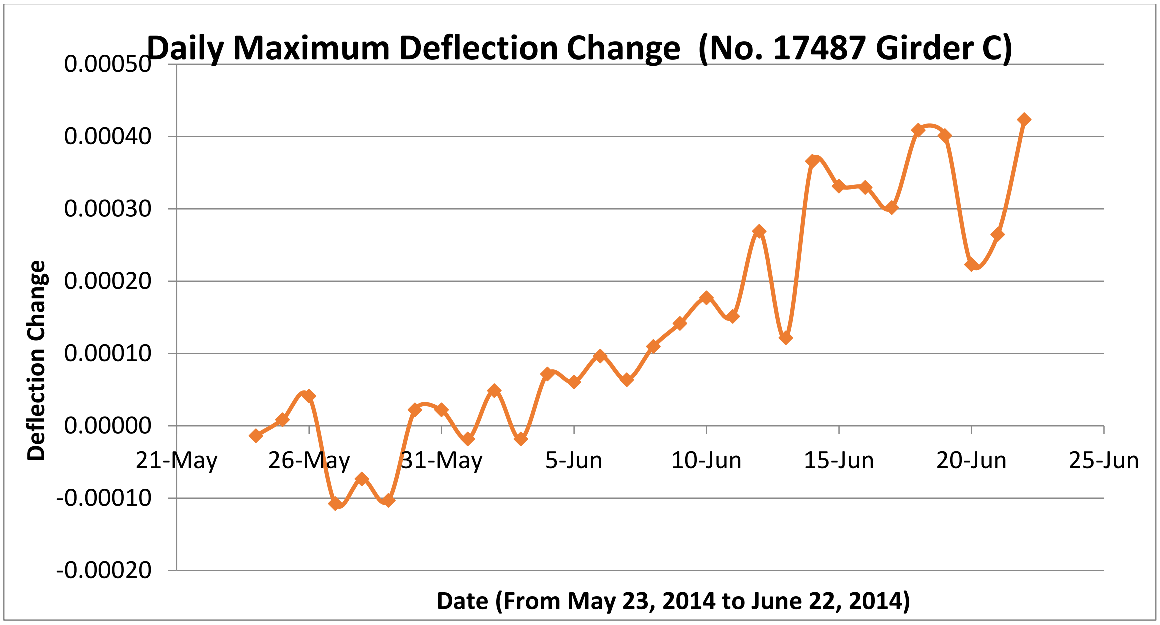

Figure 6.3: Daily maximum deflection change of Girder C.

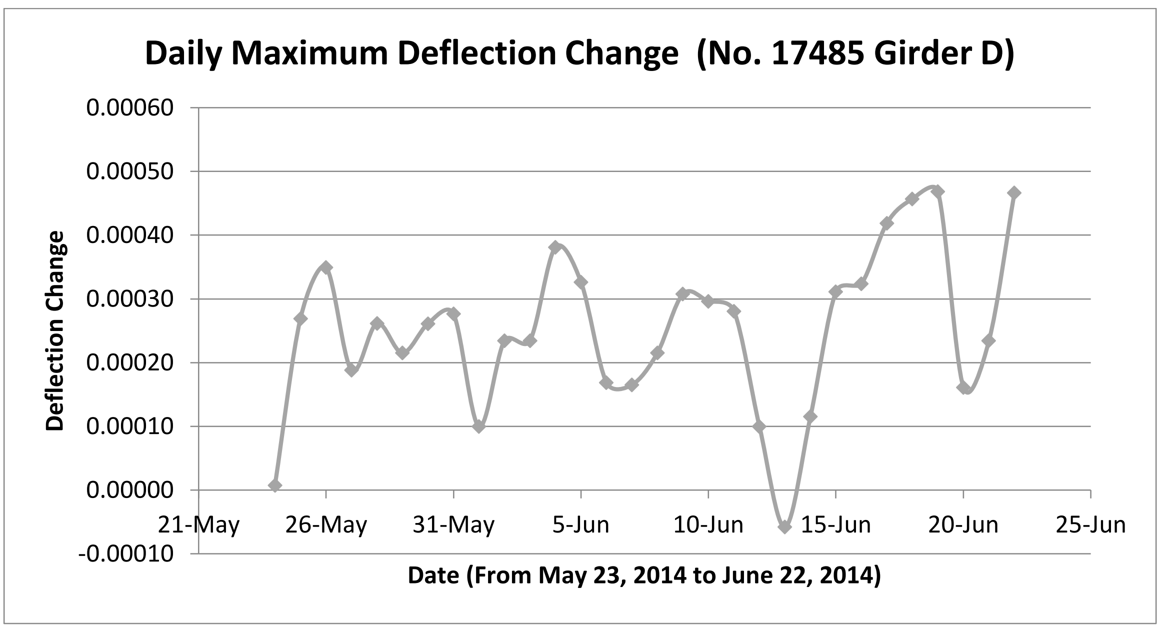

Figure 6.4: Daily maximum deflection change of Girder D.

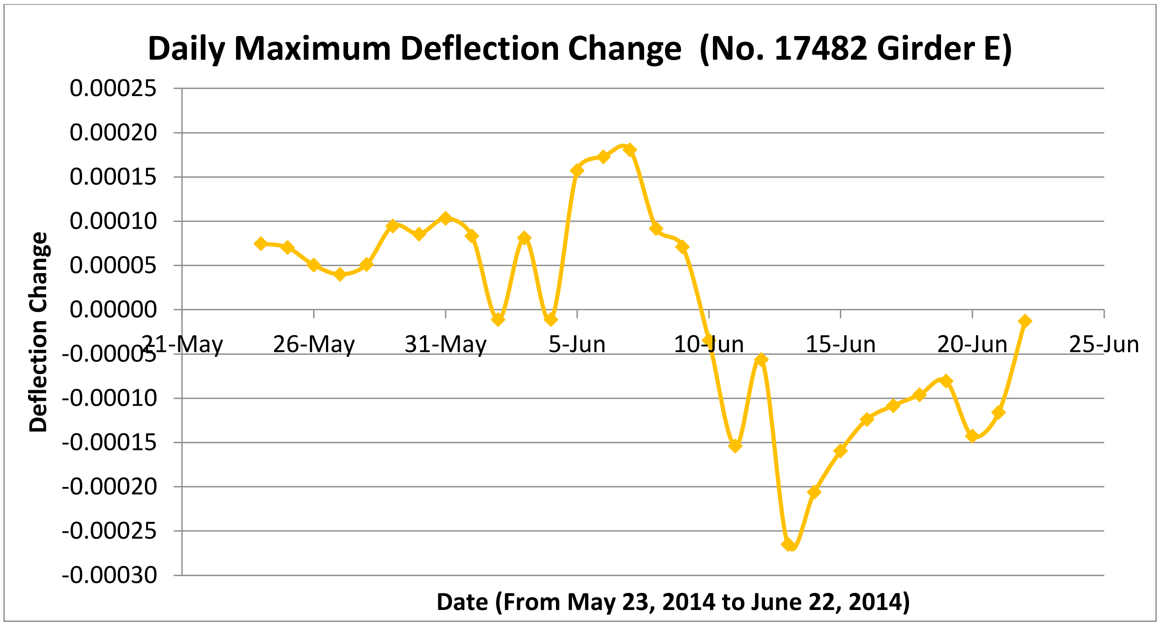

Figure 6.5: Daily maximum deflection change of Girder E.

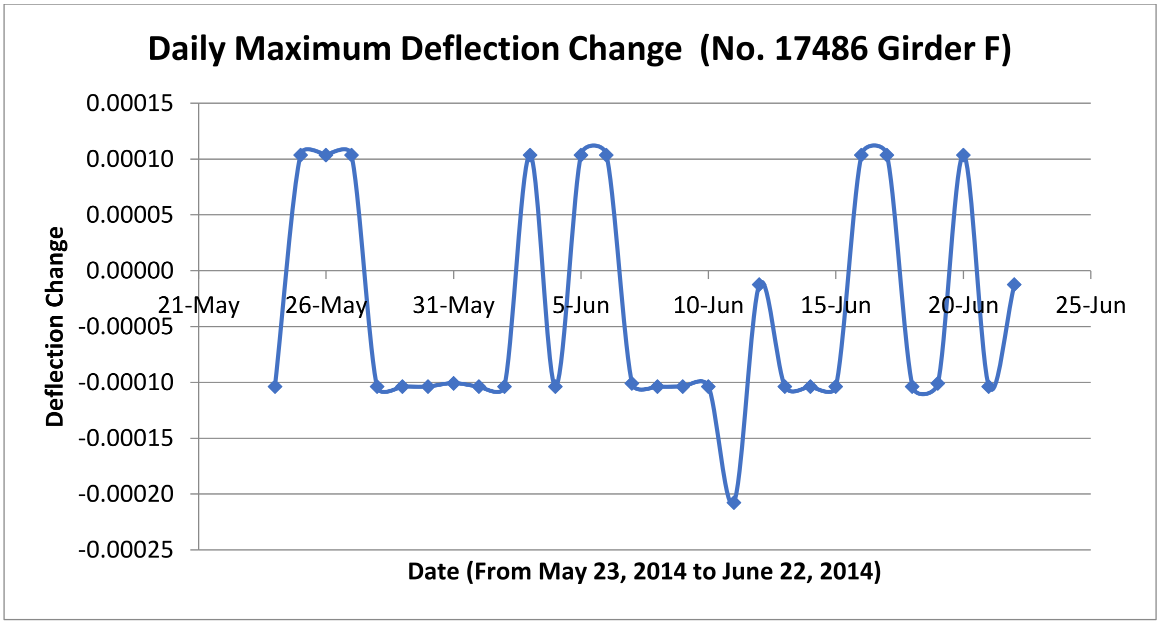

Figure 6.6: Daily maximum deflection change of Girder F.

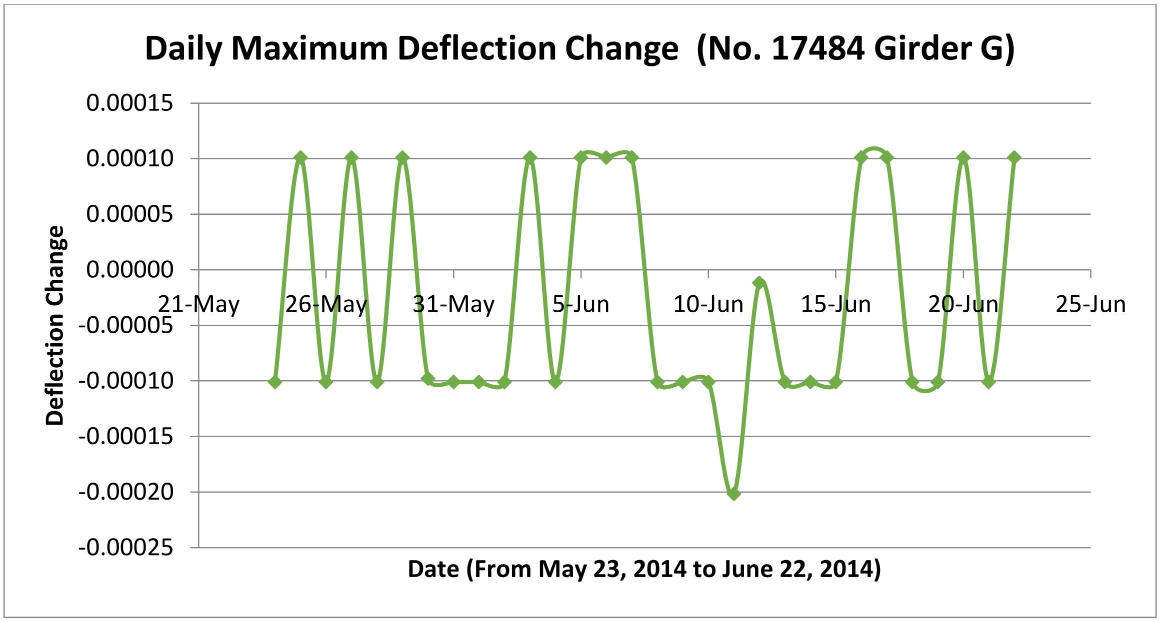

Figure 6.7: Daily maximum deflection change of Girder G.

A summary of the total deflection changes of all sensors for one month are shown in Table 6.9. Phase 2 interior girders C and D displayed larger deflections than the other monitored girders.

Table 6.9: Total Deflection Change of all Sensors for One Month

Total Deflection Change for One Month			
	Girder	Sensor No.	Deflection (in)
1	A	17480	0.00017
2	B	17478	0.00003
3	C	17487	0.00042
4	D	17485	0.00047
5	E	17482	0.00001
6	F	17486	0.00001
7	G	17484	0.00010

6.3 Girder Deflections at Closure

In this section the girder deflections of Phase 2 at time of closure pour are considered. Sensors installed on Phase 1 girders F and G were removed while closure region formwork was set. Thus, only information for the five remaining sensors are reported, before and after closure pour. The concrete closure region was poured on May 19, 2014. Therefore, data collected during the closure phase from May 15, 2014 to June 7, 2014 is presented in Figures 6.8 through 6.12. It appears that girders A, B, C, and D deformed upward at time of closure pour while girder E displaced downward.

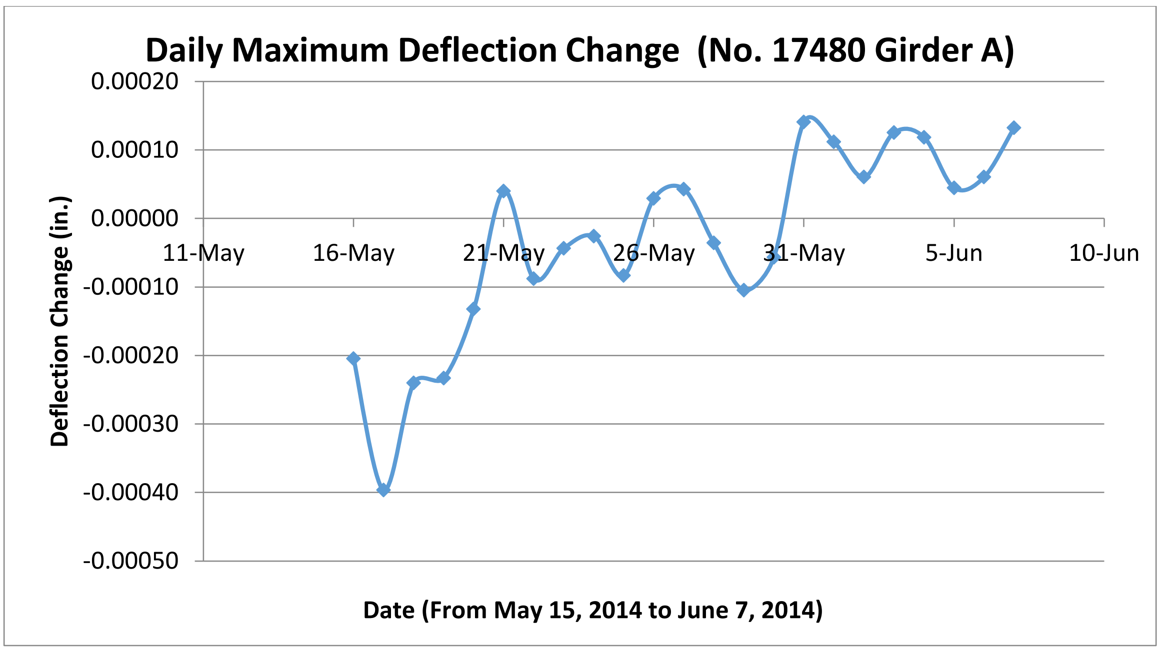

Figure 6.8: Daily maximum deflection change of Girder A.

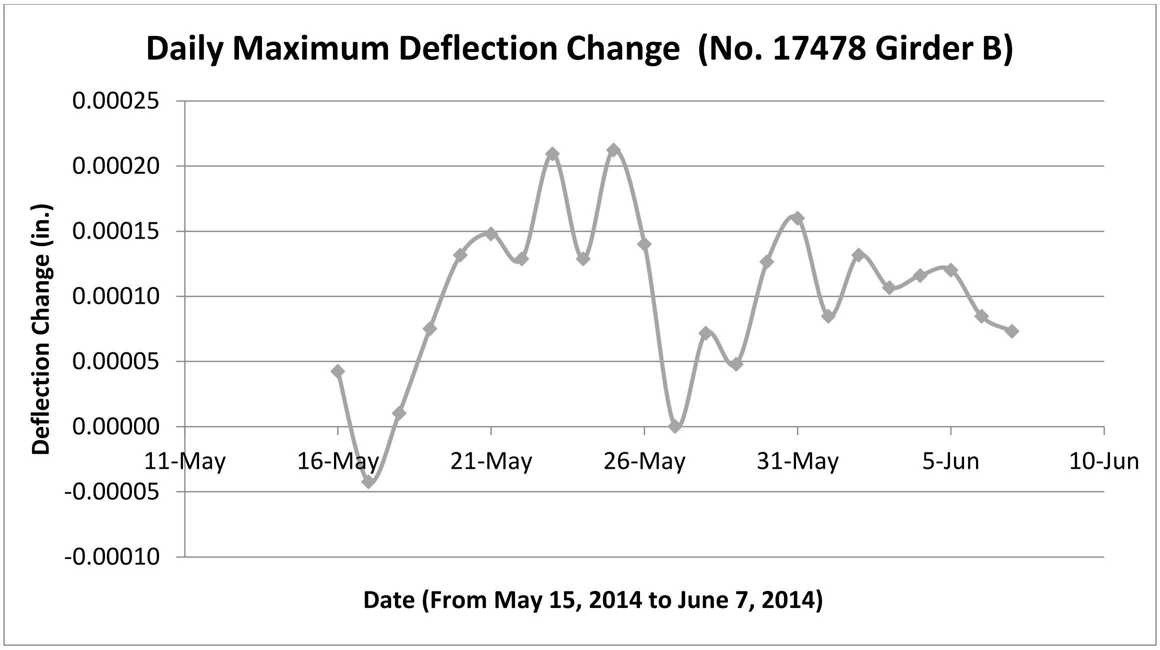

Figure 6.9: Daily maximum deflection change of Girder B.

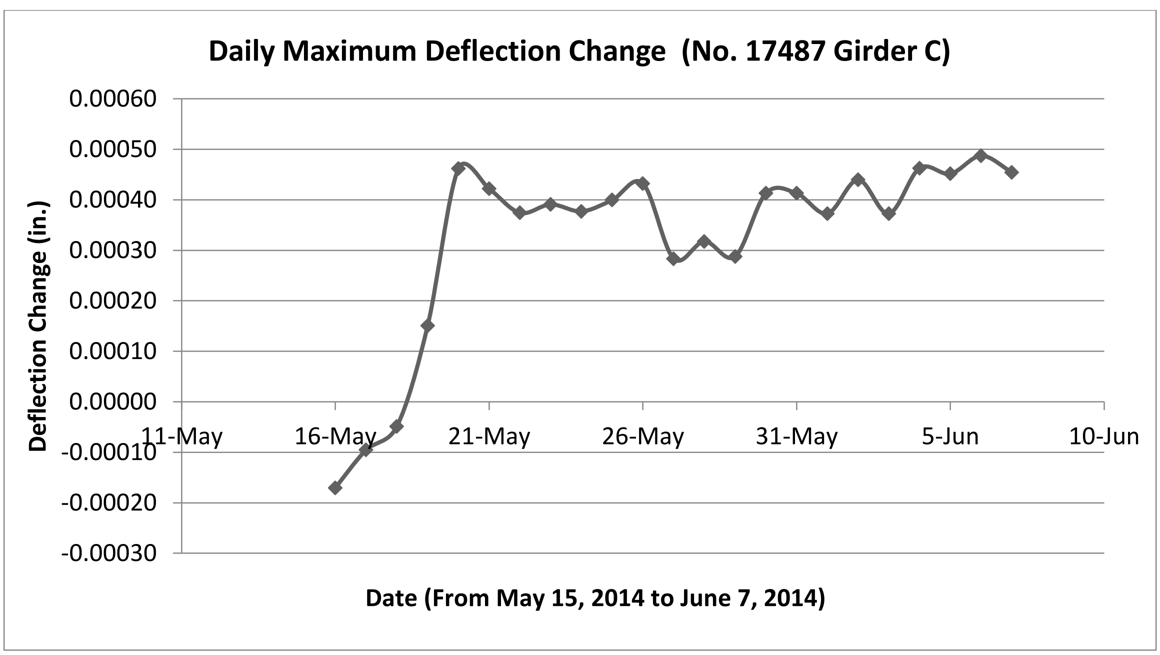

Figure 6.10: Daily maximum deflection change of Girder C.

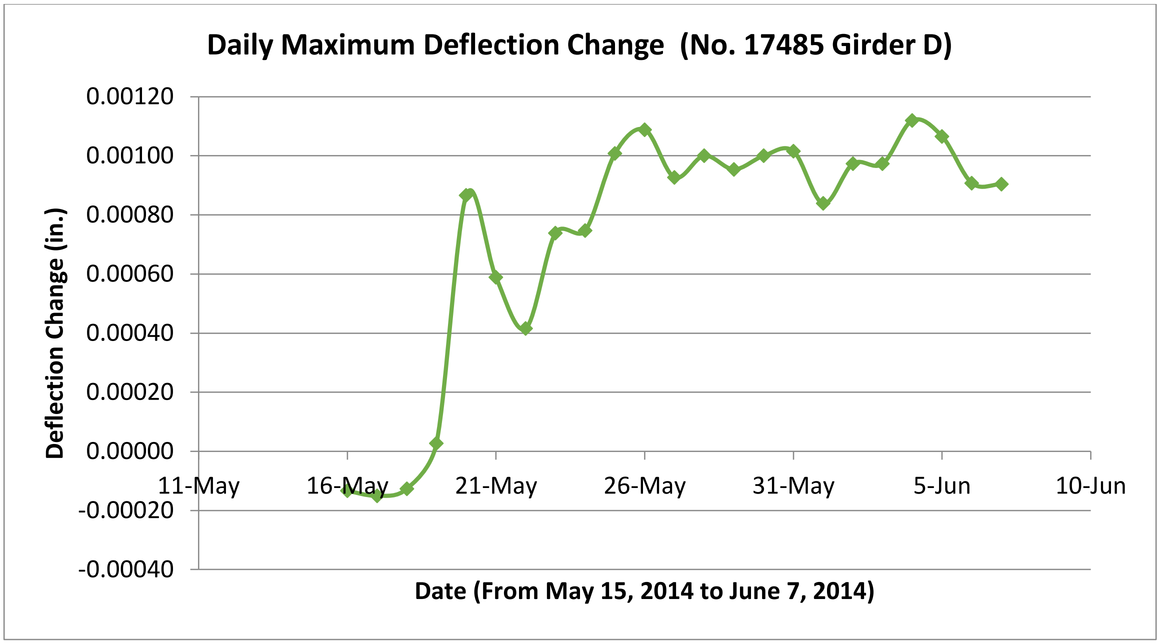

Figure 6.11: Daily maximum deflection change of Girder D.

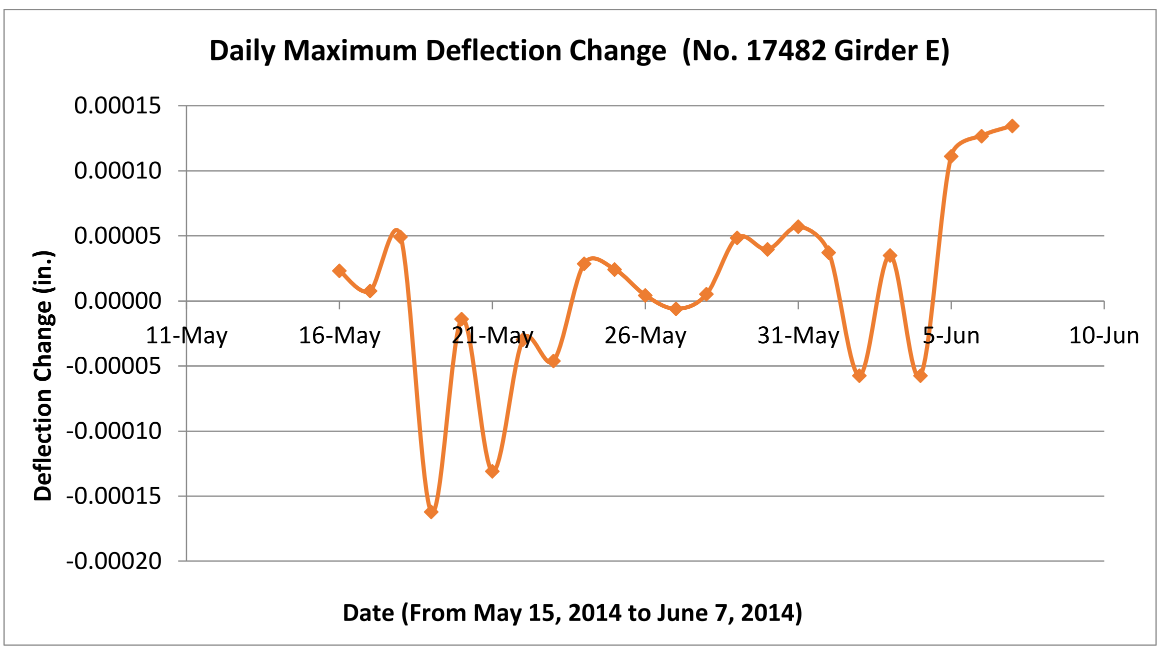

Figure 5.12: Daily maximum deflection change of Girder E.

Table 6.10 illustrates deflection difference of all girders right before and after closure pour. In this table displacement of girders on May 17 and May 21 are shown. Differences between maximum deflections for each girder in those days are calculated in this table. Girder “D” had maximum deformation upward and girder E had most downward displacement regarding to closure region concrete weight.

Table 6.10: Difference of Girders’ Displacement Before and After Closure Pour.

Deformation Difference of Girders between May 17 and May 21				
		Displacement		**Deflection**
	Girders	**17-May**	**21-May**	**Difference**
1	A	0.004514	0.004950	0.000436
2	B	0.002995	0.003185	0.000190
3	C	0.001645	0.002162	0.000517
4	D	-0.000694	0.000046	0.000740
5	E	-0.001199	-0.001337	-0.000139

6.4 Comparison of Sensor Data to Numerical Results

One of the project goals was to compare the deflection results from the displacement sensors to that of the numerical assessment and the bridge plans. However, the authors feel more evaluation of the sensor data is needed before this comparison can be completed. In addition, a software code error of the sensor data acquisition system prevented the recording of data points for the steel only deformation. Therefore the monitoring equipment did not capture the deflection information between stages of the construction process (before slab pour) of Phase 2. The data recorded and presented herein is only the maximum daily deflection.

Chapter 7. Conclusions

The goal of this project was to assess girder deflections of a phase constructed steel girder bridge being constructed in Plattsmouth, NE. The components of this study included a survey of state transportation practices, a numerical assessment and displacement monitoring during the construction process.

For the online survey administered to the State DOTs, feedback was received from 25 locations. From the survey data is was observed that the closure pour was mostly used in the Midwestern and Southeastern states. The use is typically decided on a case by case basis, deflection and span length being the deciding factors.

Dead load deflections results from the numerical model are on trend with the DL shim deflections presented in the specs. However, the values calculated have a percent difference of 20-60%. Several issues may have contributed to the difference in results. CSI Bridge uses some defaults to simplify the modeling process which provides limitations in working with the software. One limitation was having separate bent caps for each phase when the entire bridge is modeled in one code. In addition it was not possible to model two separate phases without the closure pour in one CSI Bridge model. The individual phases had to be modeled as two separate files. Moreover, the software did not allow the generations of cross frames in between girders. Its diaphragms span the entire width of the bridge. Lastly, the bridge plans present a varying degree of curvature while the model only allowed for a constant curvature along the span of the girders. Therefore more work is required to calibrate the numerical model.

In addition to a numerical assessment, the deflections of the steel girders of the Plattsmouth Bridge were monitored by EL-tilt sensors. The daily deflection changes were captured, however the maximum deflections during each step of the phased construction process were not obtained. Although the sensors were installed before the Phase 2 deck pour, data on the steel only deflections were not recorded because of software error. The Slope Indicator sensors were received without the necessary pre-installed. Therefore first week of data was not stored. In addition, several sensors were removed by contractors for closure pour formwork causing additional gaps in collected data. Lastly, the maximum daily deflections presented herein are not directly comparable to the maximum total deflections of the plans or numerical model. More evaluation of the monitoring data is needed to complete the results comparison. Thus, the causes for the differential elevation have yet to be determined.

References:

AASHTO, (2003). “Guidelines for Design for Constructability” AASHTO/ NSBA Steel Bridge Collaboration. G. 12.1.

American Association of State Highway and Transportation Officials Executive Committee (AASHTO)/ National Steel Bridge Alliance (NSBA); “Guidelines for Steel Girder Bridge Analysis” 1st Edition; 2010-2011

Azizinamini, A., Yakel, A., and Swendroski, J. (2003). “Development of a Design Guideline for Phase Construction of Steel Girder Bridges” NDOR Final Report, Project Number SPR-PL-1 (038) P530, October 2003.

CSI. (2011). “CSI Bridge Advanced Software version 15”. Computers & Structures, Inc.

Nebraska Department of Roads. (2004). “Wood River Bridge Construction” NDOR Report, Project Number S080-30013, November 2004.

Nebraska Department of Roads. (2008). “I-80 Hershey Interchange” NDOR Report, Project Number S-80-3(1039), November 2008.

Nebraska Department of Roads (2013). “Bridge Office Policies and Procedures (BOPP)” Bridge Division Manual, 2013.

Retterer, T. (2012). “Phased Construction Recommendations for Bridges” BRG Webinar for Texas Department of Transportation, July 2012.

Taveras, M., Lourdes, A., "Dynamic Testing and Finite Element Modeling of a Steel Girder Bridge for the Long-Term Bridge Performance Program" (2012). *All Graduate Theses and Dissertations.* Paper 1229.

Yakel, A., Marchon, P., and Azizinamini, A.; “Long Term Monitoring of a Steel Bridge Constructed Using Phase Construction”; Nebraska Bridge Research Organization (NaBRO), Department of Civil Engineering; (July, 2005).

Appendix A

A.1 CSI Bridge Modeling Steps

In order to understand CSI Bridge program details and steps of Phase 2 modeling are indicated as follow:

A.1.1 Layout Line Data

Initial Station and End Station are -70ft and 552ft respectively (to show better the skew of abutments and curve of bridge). (This model is just for one phase)

i. For horizontal curve “Curve Right” in “Quick Start” is chosen; Figure A.1.

ii. The radius is 6258.7ft from plan with “S860000E” in Bearing PI to EC; Figure A.2.

i. Because this model is just for one Phase, in Bridge lane data the information of one phase is generated. Center line offset is (-37.125ft) for phase 1 (for phase 2 is (+37.125)) and Lane width is (63.75ft); Figure A.3.

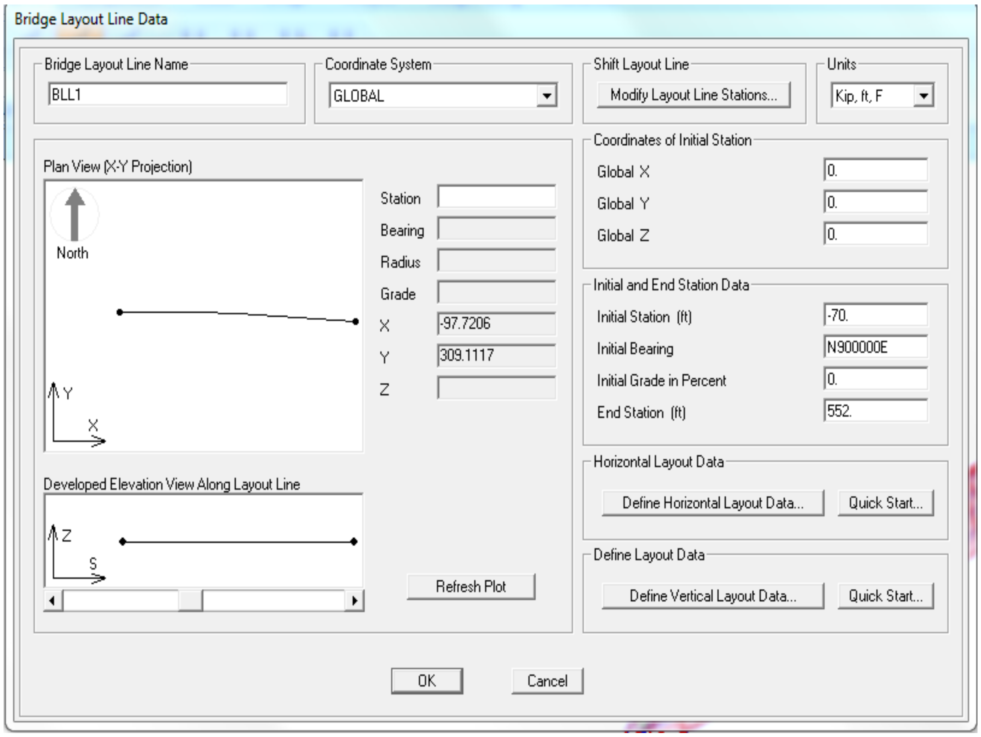

Figure A.1

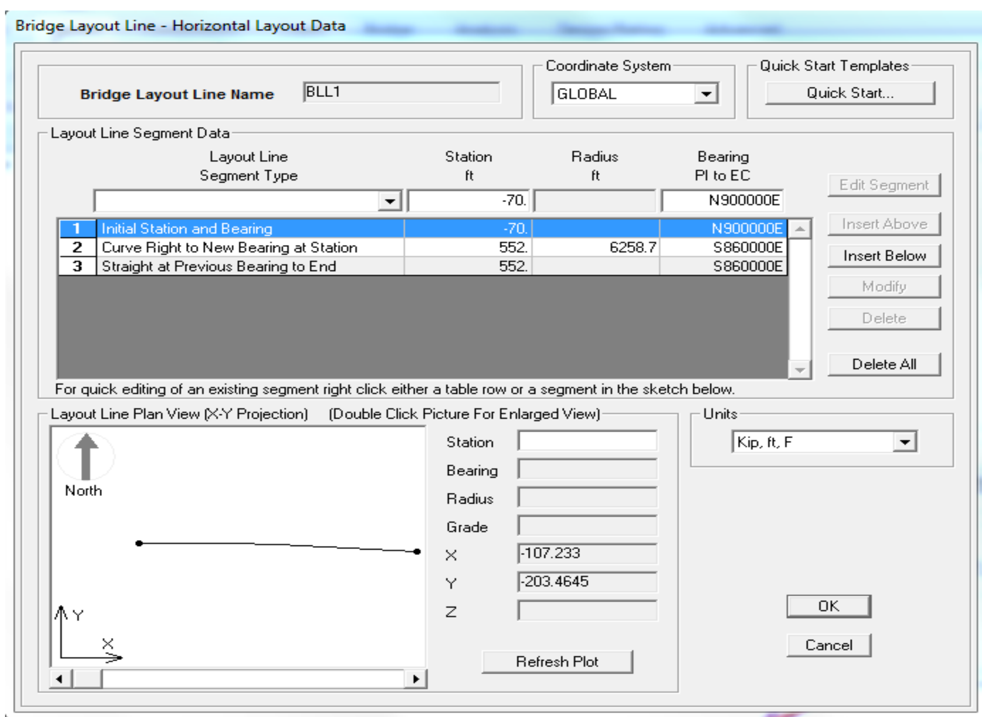

Figure A.2

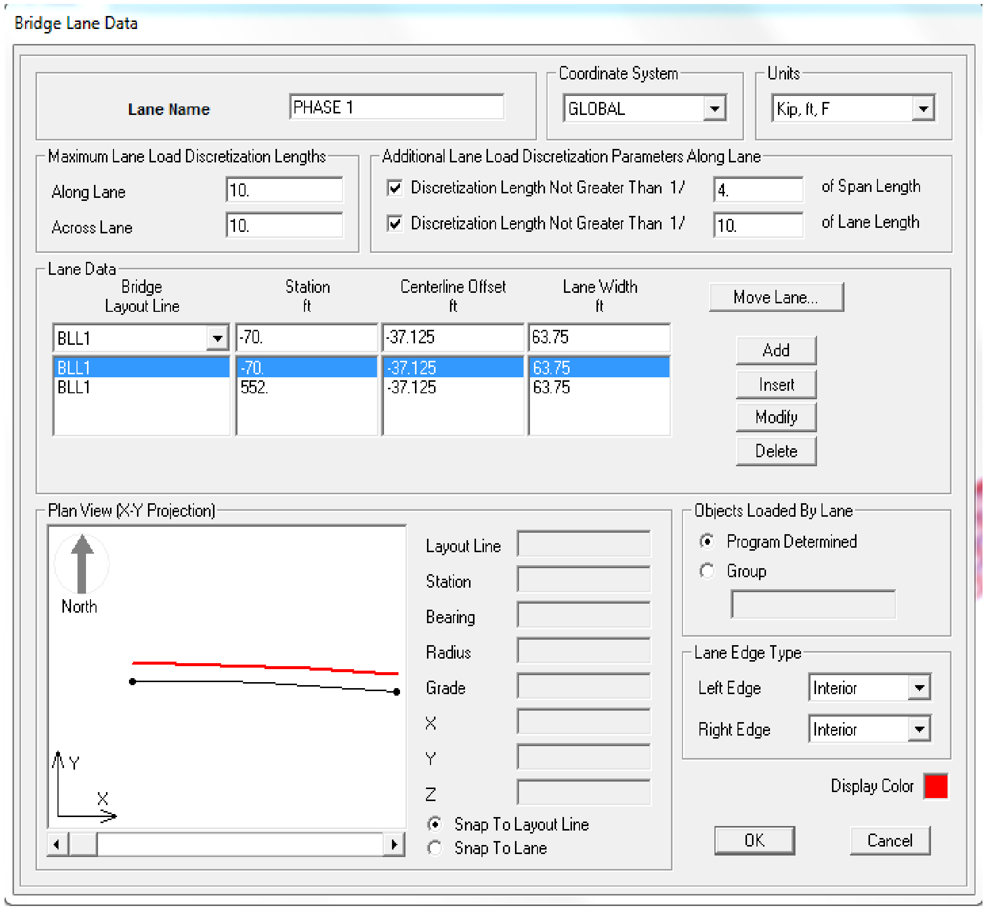

Figure A.3

A.1.2 Components

Properties – Frames: Components are defined as follow: all Components concrete material is 3000psi and slab material is 4000psi.

a. Abutment is shown in Figure A.4 Number of longitudinal bars along 3-dir face is 5 and along 2-dir face is 7, bars size is #9; Figure A.5.

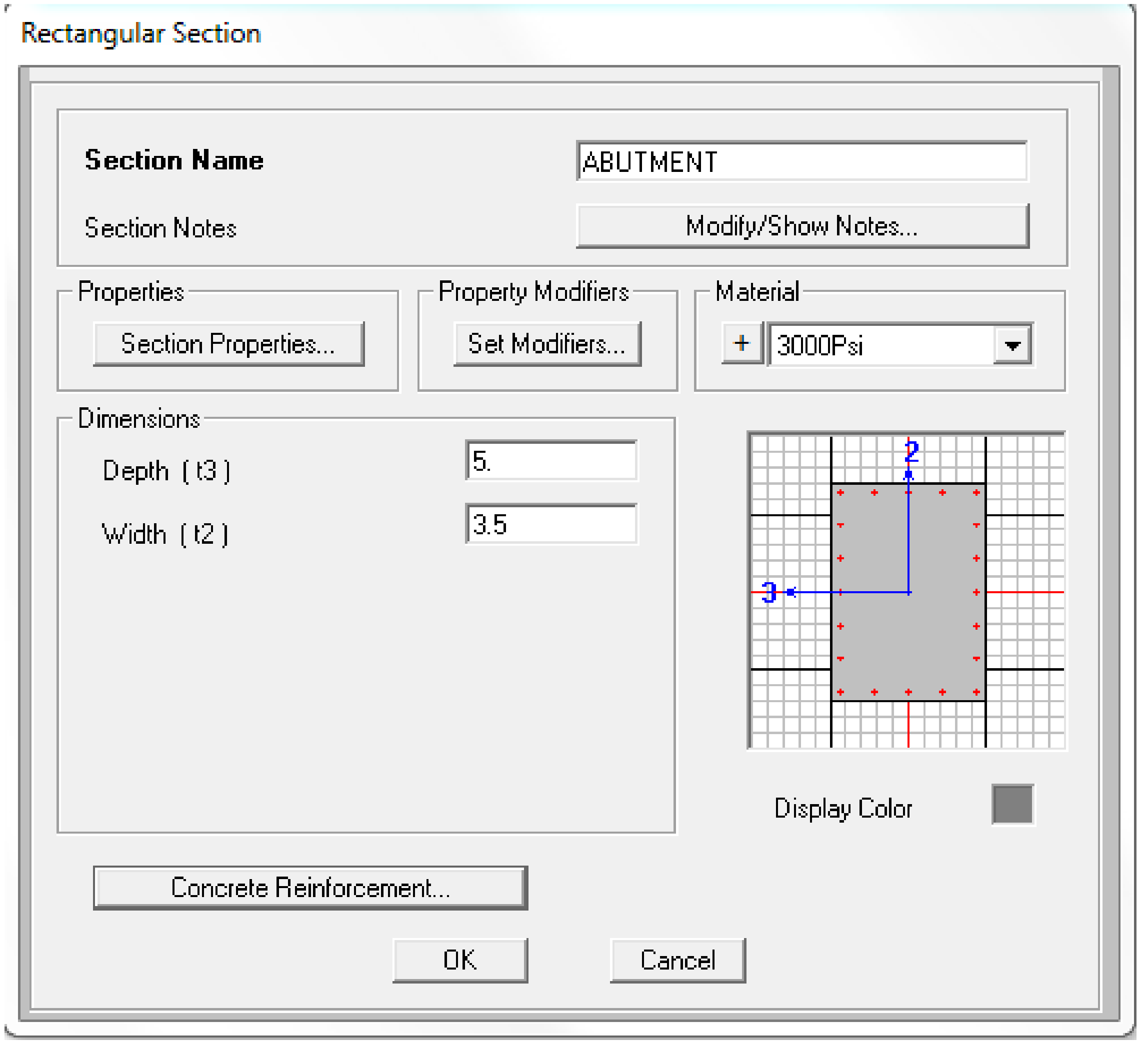
Rectangular Section
Section Name
ABUTMENT
Section Notes
Modify/Show Notes...
Properties
Section Properties...
Property Modifiers
Set Modifiers...
Material
+
3000Psi
Dimensions
Depth (t3)
5.
Width (t2)
3.5
2
3
Display Color
Concrete Reinforcement...
OK
Cancel

Figure A.4

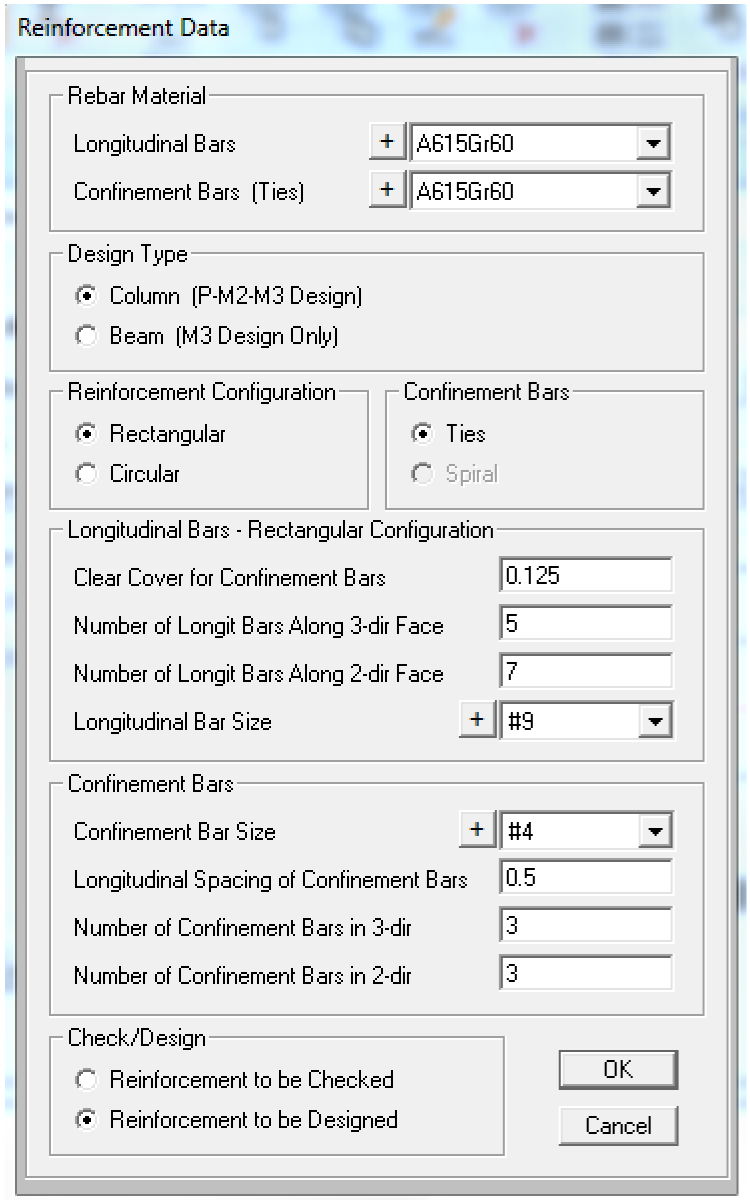
Reinforcement Data
Rebar Material
Longitudinal Bars
A615Gr60
Confinement Bars (Ties)
A615Gr60
Design Type
Column (P-M2-M3 Design)
Beam (M3 Design Only)
Reinforcement Configuration
Rectangular
Circular
Confinement Bars
Ties
Spiral
Longitudinal Bars - Rectangular Configuration
Clear Cover for Confinement Bars
0.125
Number of Longit Bars Along 3-dir Face
5
Number of Longit Bars Along 2-dir Face
7
Longitudinal Bar Size
#9
Confinement Bars
Confinement Bar Size
#4
Longitudinal Spacing of Confinement Bars
0.5
Number of Confinement Bars in 3-dir
3
Number of Confinement Bars in 2-dir
3
Check/Design
Reinforcement to be Checked
Reinforcement to be Designed
OK
Cancel

Figure A.5

b. Column is shown in Figure A.6. Number of longitudinal bars along 3-dir face is 9 and along 2-dir face is 5, bars size is #9; Figure A.7.

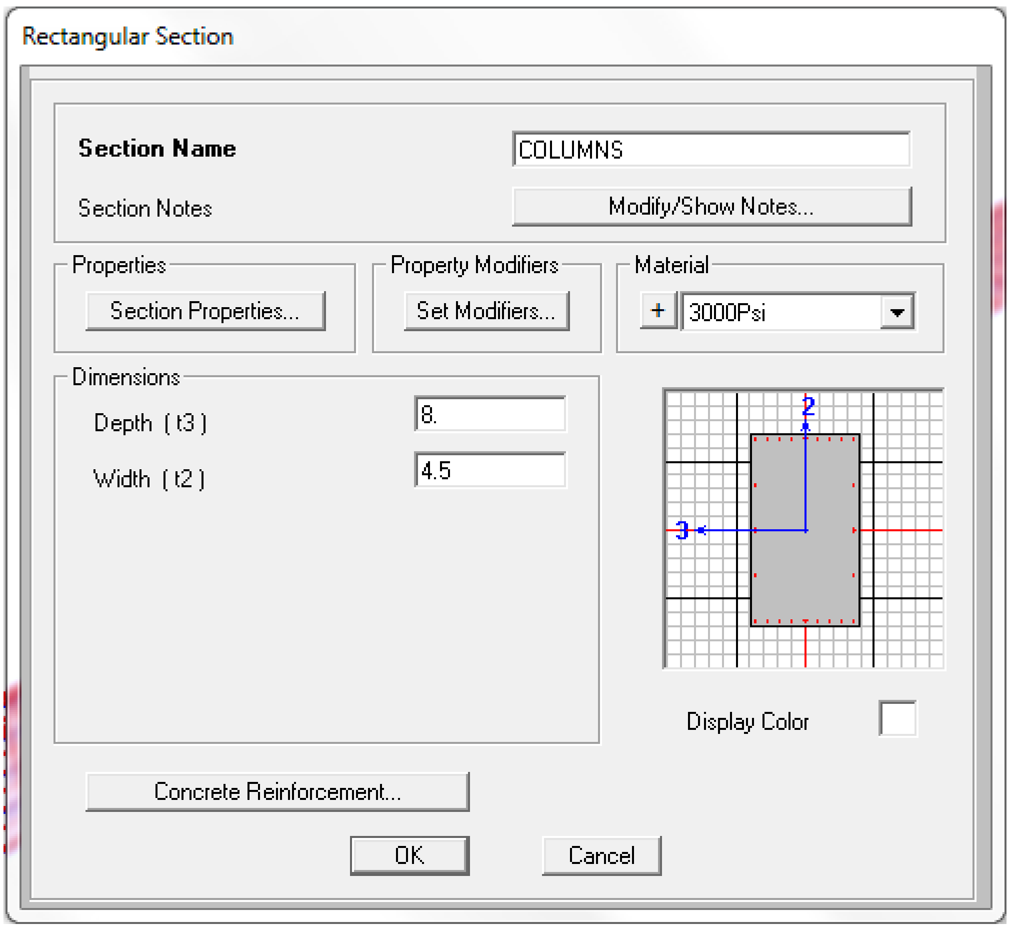

Figure A.6

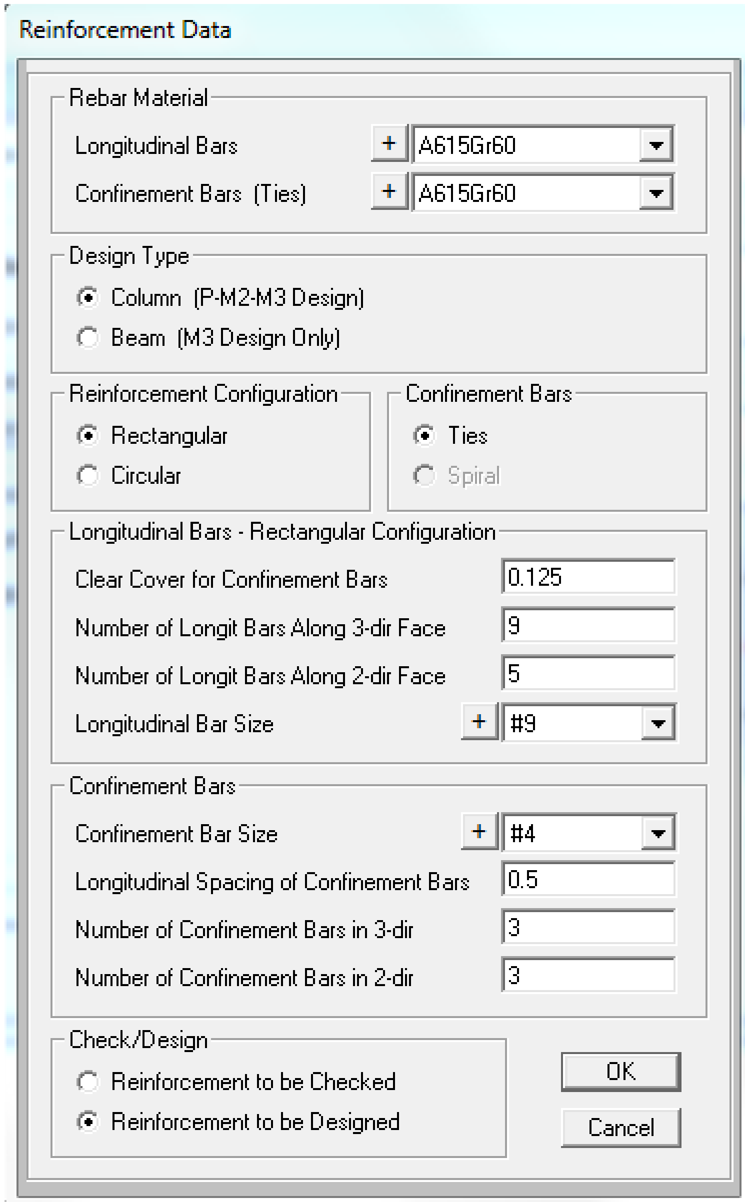
Reinforcement Data
Rebar Material
Longitudinal Bars
A615Gr60
Confinement Bars (Ties)
A615Gr60
Design Type
Column (P-M2-M3 Design)
Beam (M3 Design Only)
Reinforcement Configuration
Rectangular
Circular
Confinement Bars
Ties
Spiral
Longitudinal Bars - Rectangular Configuration
Clear Cover for Confinement Bars
0.125
Number of Longit Bars Along 3-dir Face
9
Number of Longit Bars Along 2-dir Face
5
Longitudinal Bar Size
#9
Confinement Bars
Confinement Bar Size
#4
Longitudinal Spacing of Confinement Bars
0.5
Number of Confinement Bars in 3-dir
3
Number of Confinement Bars in 2-dir
3
Check/Design
Reinforcement to be Checked
Reinforcement to be Designed
OK
Cancel

Figure A.7

c. Steel Girders are A992 with Fy50. The height is 4.833ft, top and bottom flange width and thickness in addition of web thickness are vary in different areas; Figure A.8.

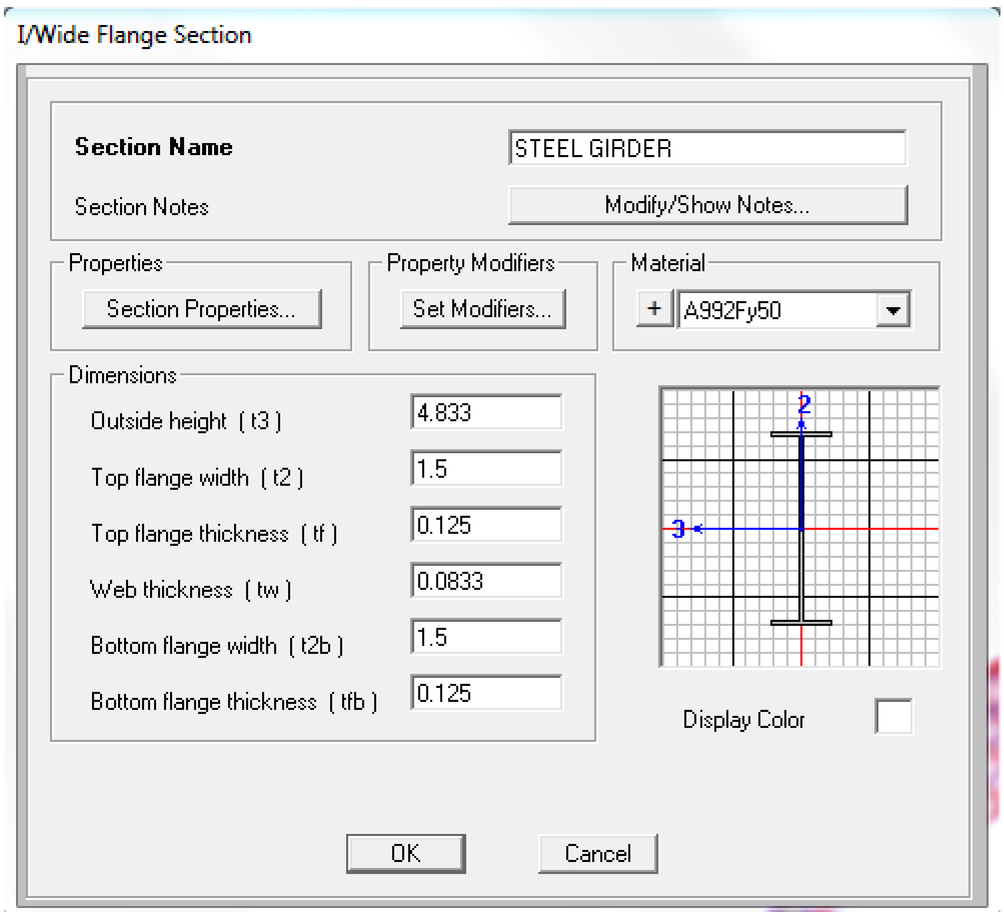

Figure A.8

d. Bent Cap is shown in Figure A.9. Number of longitudinal bars along 3-dir face is 8 and along 2-dir face is 6, bars size is #9; Figure A.10.

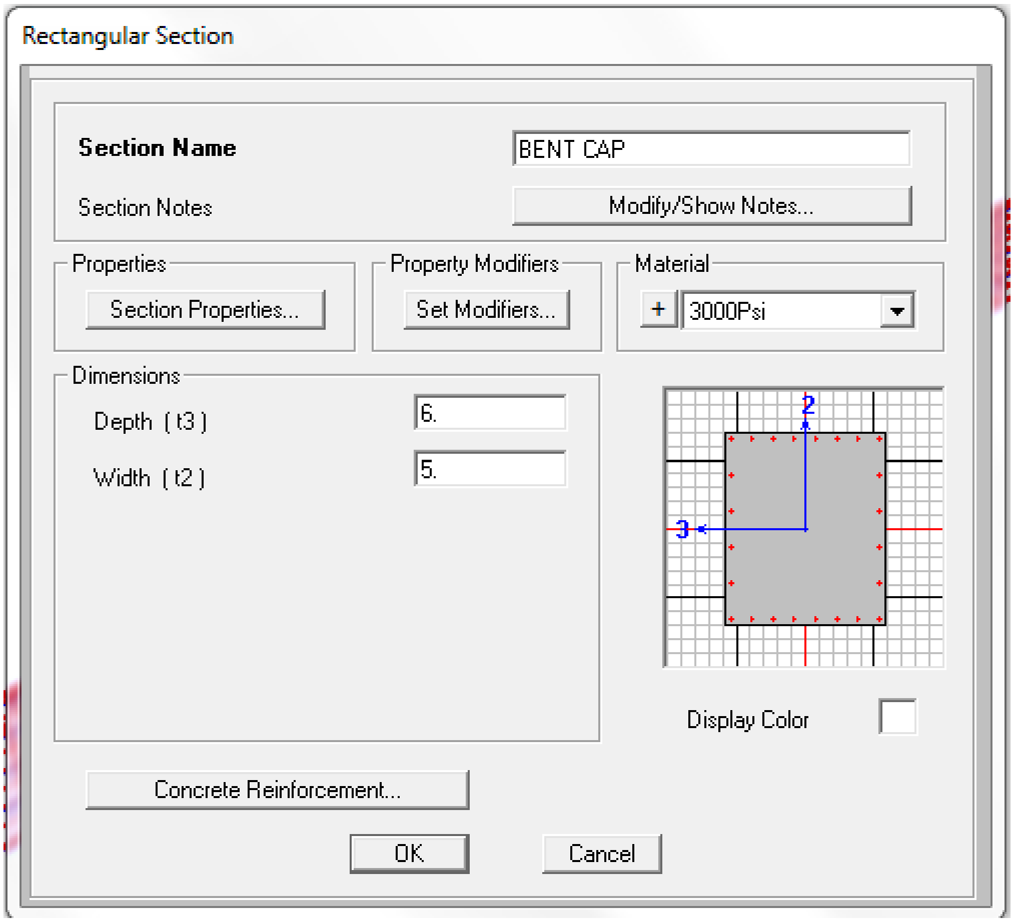

Figure A.9

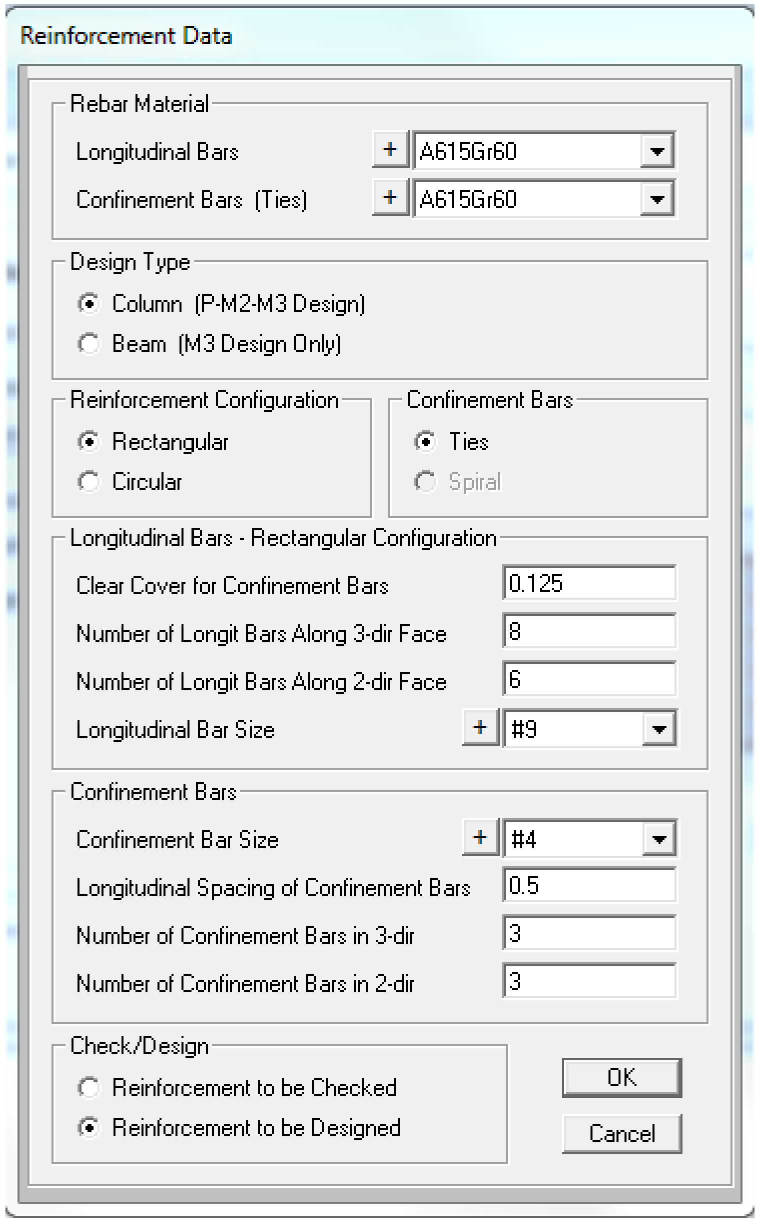
Reinforcement Data
Rebar Material
Longitudinal Bars
A615Gr60
Confinement Bars (Ties)
A615Gr60
Design Type
Column (P-M2-M3 Design)
Beam (M3 Design Only)
Reinforcement Configuration
Rectangular
Circular
Confinement Bars
Ties
Spiral
Longitudinal Bars - Rectangular Configuration
Clear Cover for Confinement Bars
0.125
Number of Longit Bars Along 3-dir Face
8
Number of Longit Bars Along 2-dir Face
6
Longitudinal Bar Size
#9
Confinement Bars
Confinement Bar Size
#4
Longitudinal Spacing of Confinement Bars
0.5
Number of Confinement Bars in 3-dir
3
Number of Confinement Bars in 2-dir
3
Check/Design
Reinforcement to be Checked
Reinforcement to be Designed
OK
Cancel

Figure A.10

A.1.3 Superstructure

a. Deck Sections: Slab thickness is 8in, total width of slab is 45.0833ft. Thickness of Haunch plus flange is 4.25in.; Figure A.11 and A.12.

b. Bridge diaphragm property is shown in Figure A.13.

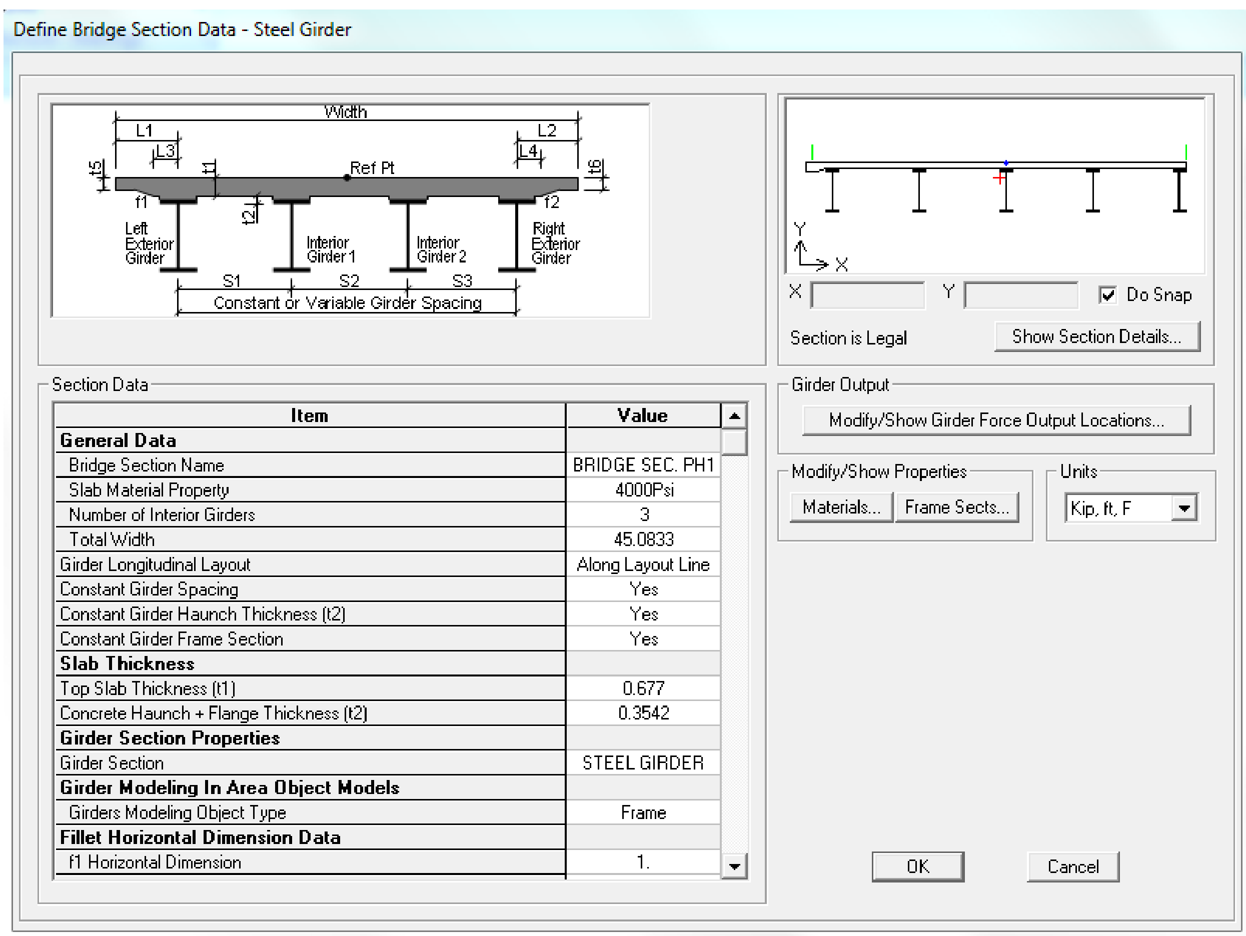

Figure A.11

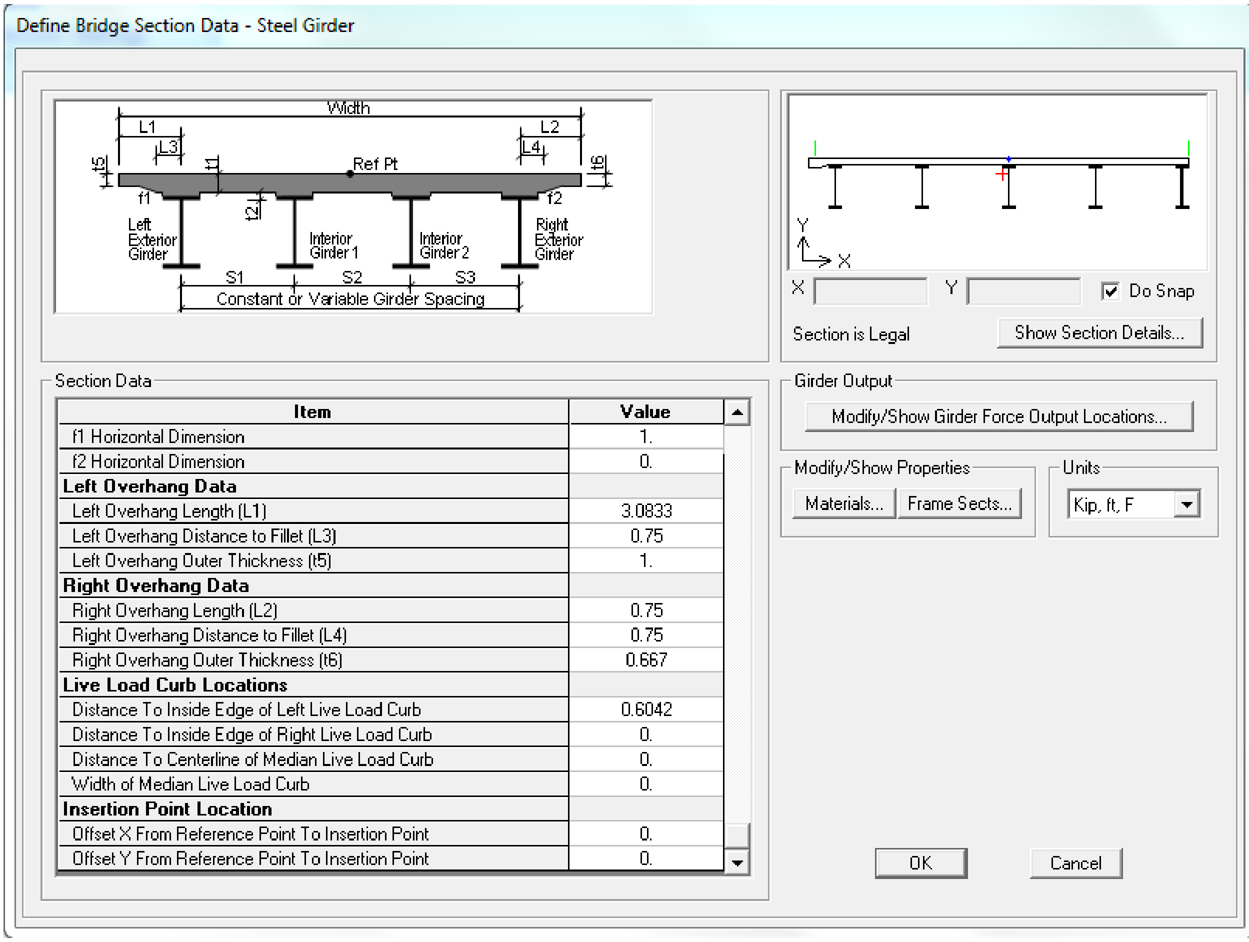
Define Bridge Section Data - Steel Girder
Width
L1
L3
L2
L4
t5
t1
t6
Ref Pt
f1
f2
t2
Left Exterior Girder
Interior Girder 1
Interior Girder 2
Right Exterior Girder
S1
S2
S3
Constant or Variable Girder Spacing
Y
X
X
Y
Do Snap
Section is Legal
Show Section Details...
Section Data
Item
Value
f1 Horizontal Dimension
1.
f2 Horizontal Dimension
0.
Left Overhang Data
Left Overhang Length (L1)
3.0833
Left Overhang Distance to Fillet (L3)
0.75
Left Overhang Outer Thickness (t5)
1.
Right Overhang Data
Right Overhang Length (L2)
0.75
Right Overhang Distance to Fillet (L4)
0.75
Right Overhang Outer Thickness (t6)
0.667
Live Load Curb Locations
Distance To Inside Edge of Left Live Load Curb
0.6042
Distance To Inside Edge of Right Live Load Curb
0.
Distance To Centerline of Median Live Load Curb
0.
Width of Median Live Load Curb
0.
Insertion Point Location
Offset X From Reference Point To Insertion Point
0.
Offset Y From Reference Point To Insertion Point
0.
Girder Output
Modify/Show Girder Force Output Locations...
Modify/Show Properties
Materials...
Frame Sects...
Units
Kip, ft, F
OK
Cancel

Figure A.12

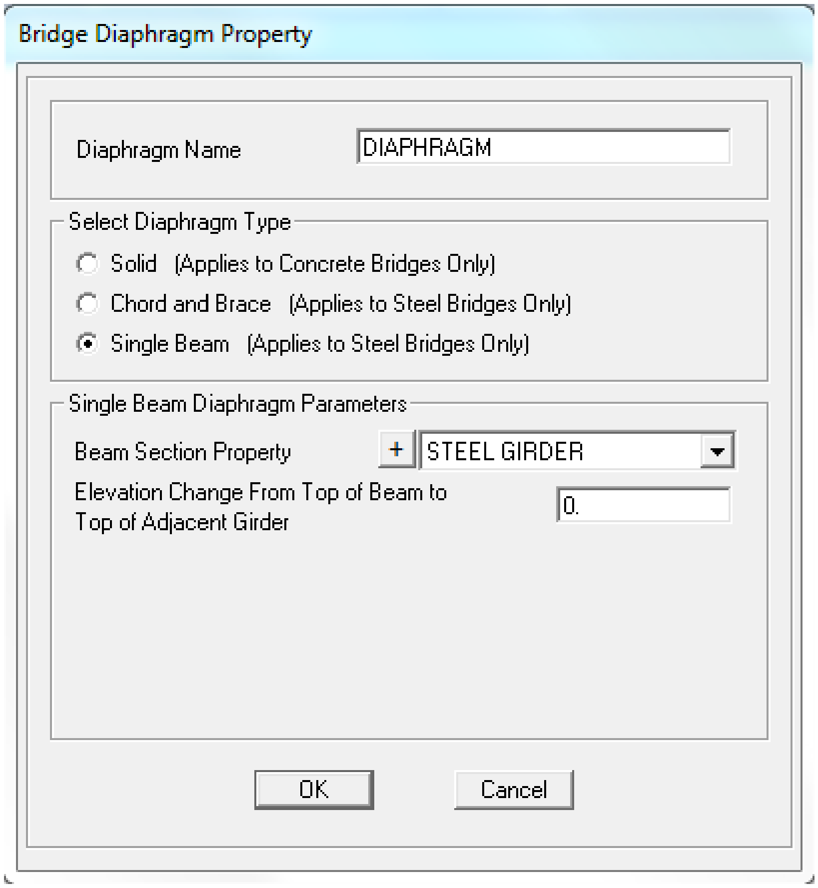
Bridge Diaphragm Property
Diaphragm Name
DIAPHRAGM
Select Diaphragm Type
Solid (Applies to Concrete Bridges Only)
Chord and Brace (Applies to Steel Bridges Only)
Single Beam (Applies to Steel Bridges Only)
Single Beam Diaphragm Parameters
Beam Section Property
STEEL GIRDER
Elevation Change From Top of Beam to Top of Adjacent Girder
0.
OK
Cancel

Figure A.13

A.1.4 Substructure

a. Bearing data and degree of freedom is shown in Figure A.14.

b. There are two bents for bridge. Bents data and columns for each bent are indicated in Figures A.15 and A.16 (For bent 1) and Figures A.17 and A.18 (For bent 2).

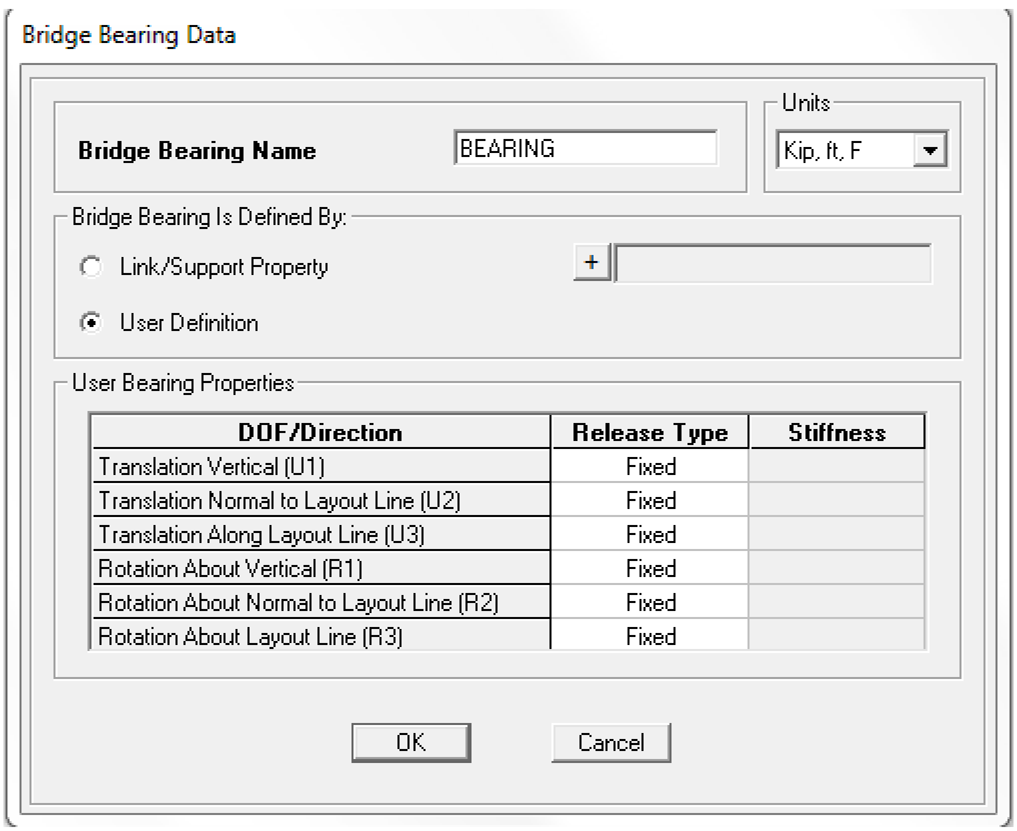

Figure A.14

Figure A.15 (Bent 1)

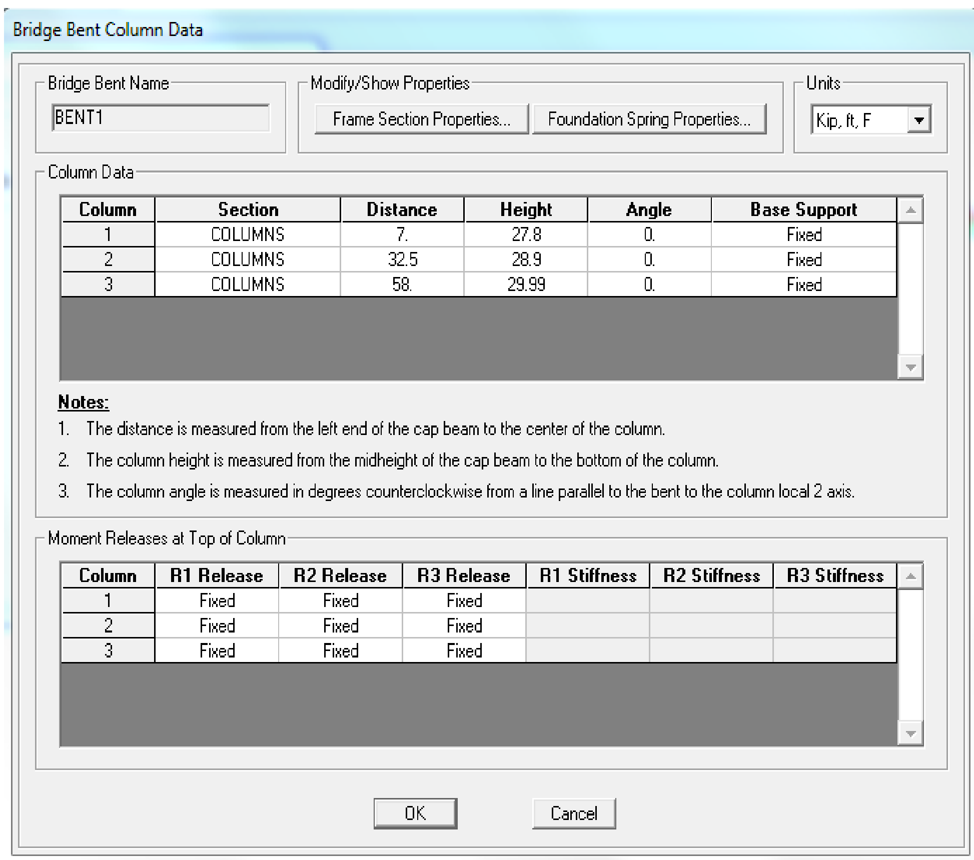

Figure A.16 (Column of Bent 1)

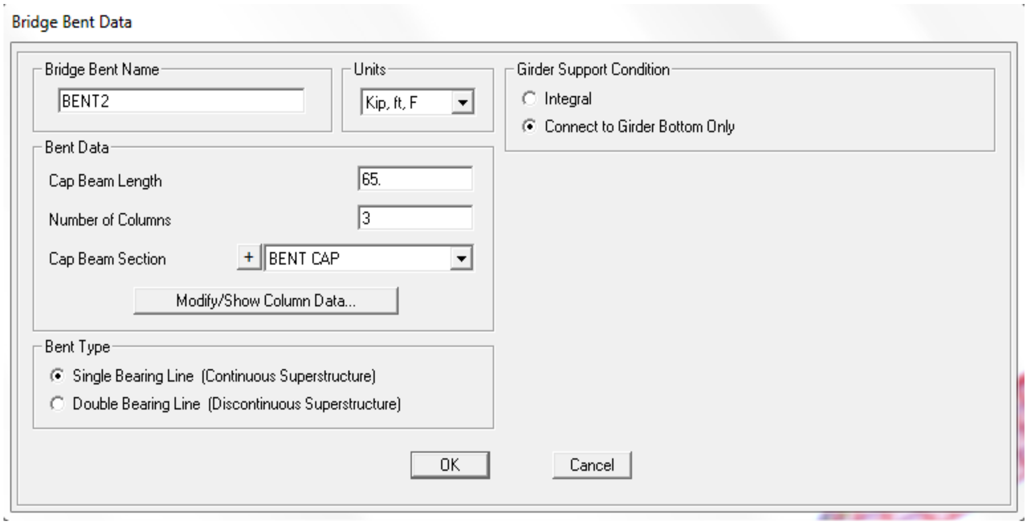

Figure A.17 (Bent 2)

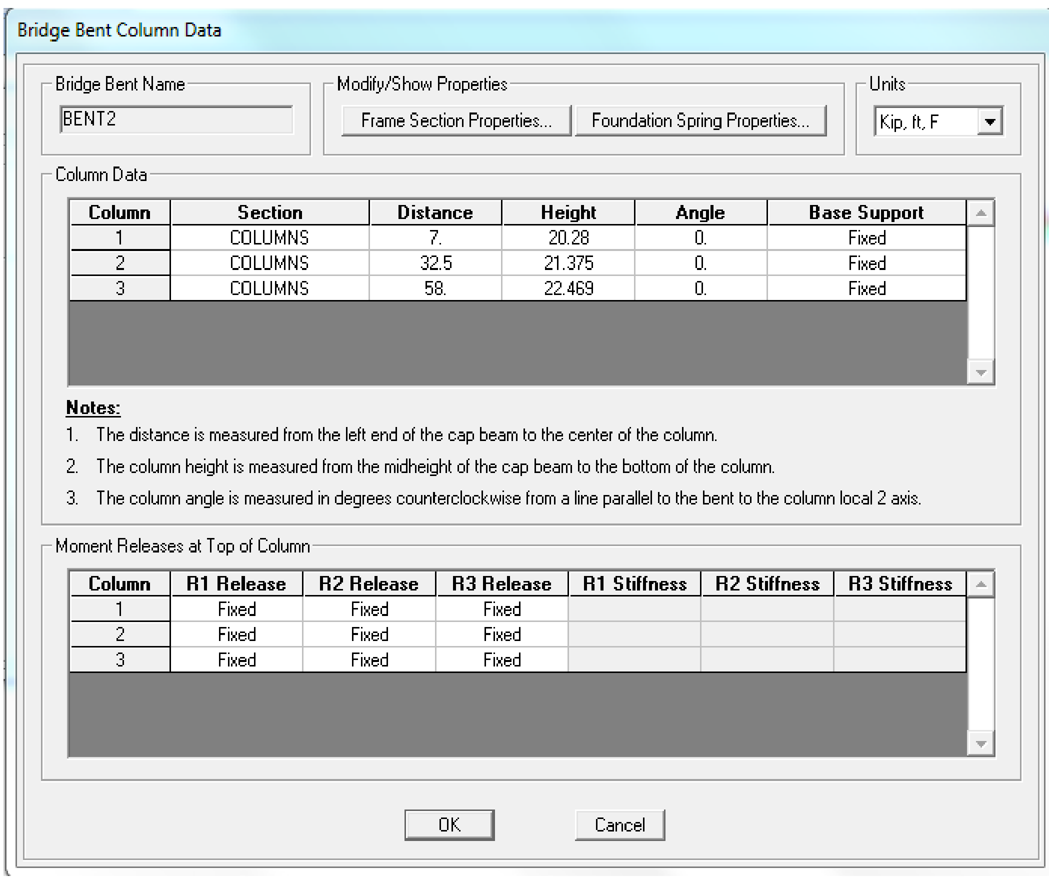

Figure A.18 (Columns of Bent 2)

A.1.5 Load

a. Vehicle Data: Vehicle type is HSn-44L; Figure A.19. (It is not applied for our model)

b. Load pattern is shown in Figure A.20.

c. Live (Temporary Barrier) is defined as line load; Figure A.21.

d. Moving Load is defined as area load. Its area load distribution is shown in Figure A.22.

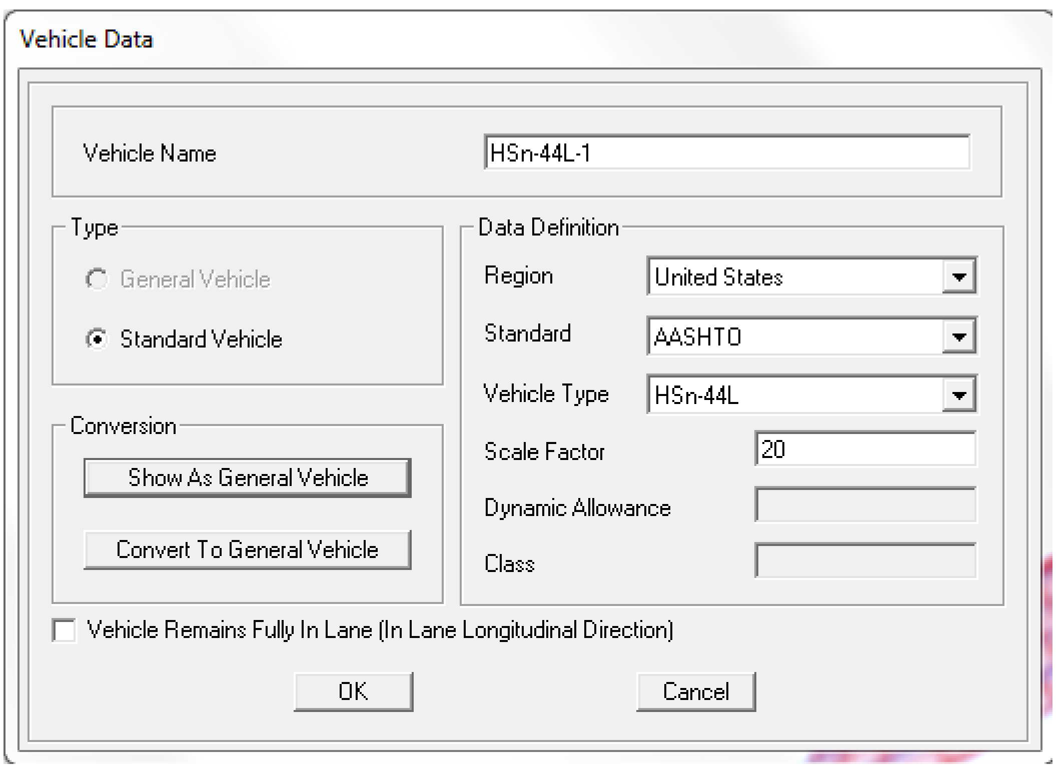

Figure A.19

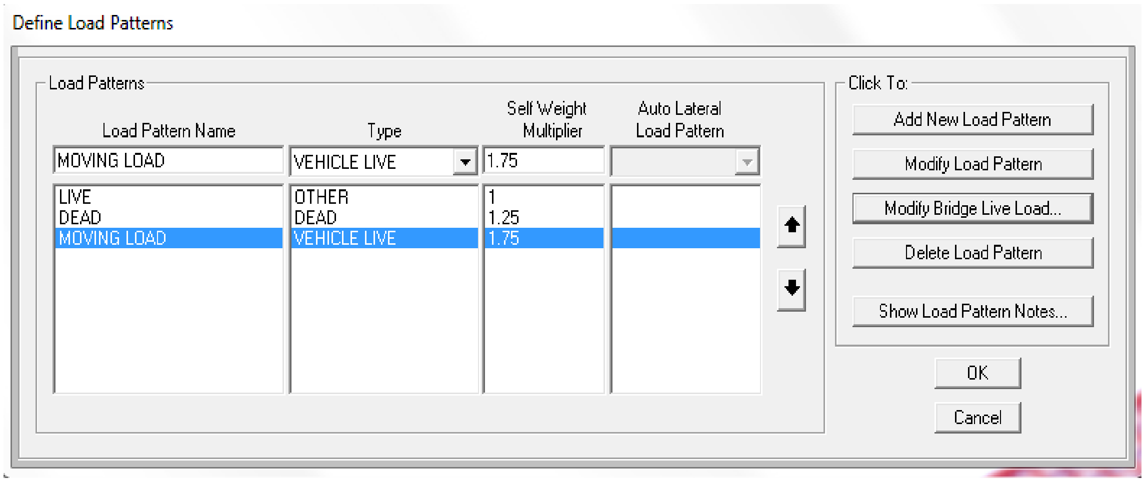

Figure A.20

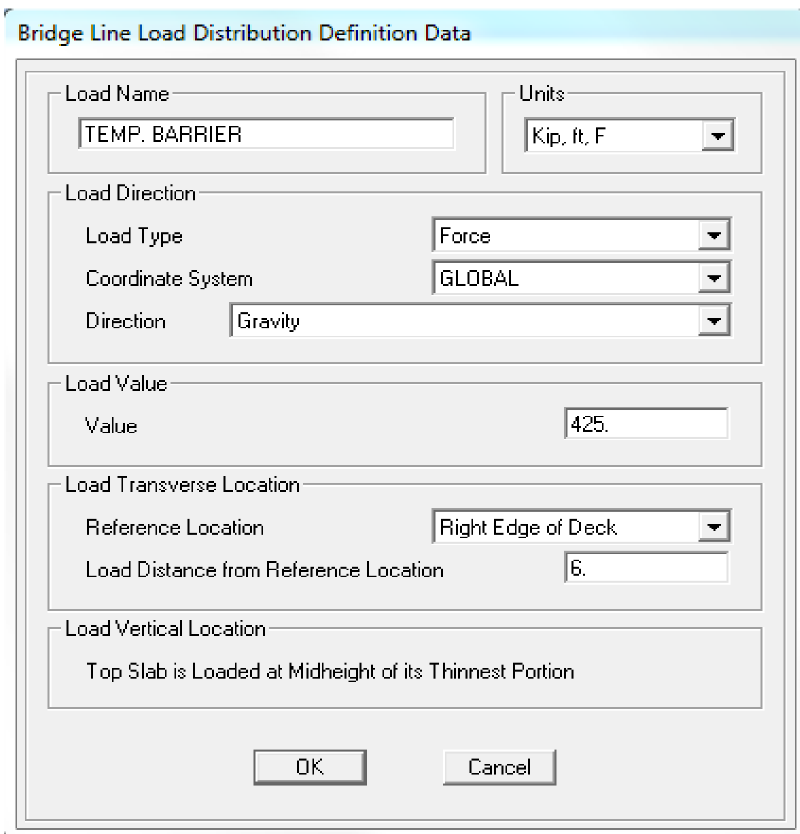

Figure A.21

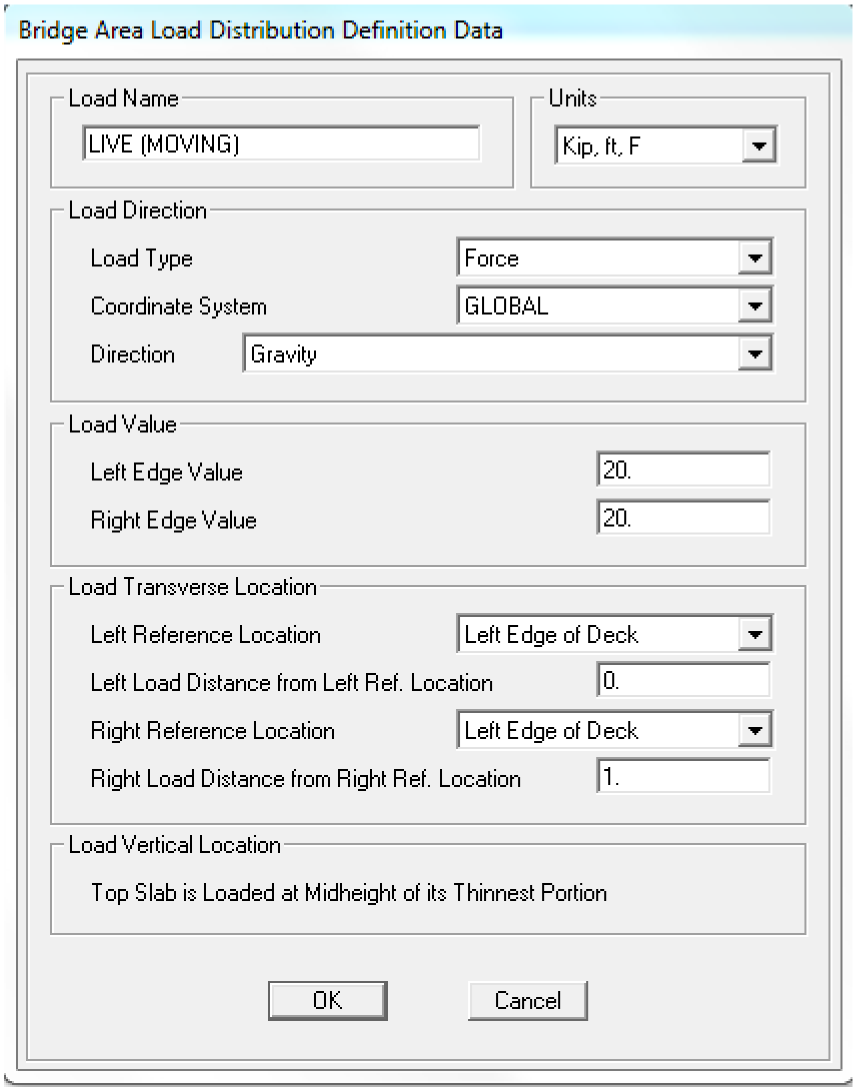
Bridge Area Load Distribution Definition Data
Load Name
LIVE (MOVING)
Units
Kip, ft, F
Load Direction
Load Type
Force
Coordinate System
GLOBAL
Direction
Gravity
Load Value
Left Edge Value
20.
Right Edge Value
20.
Load Transverse Location
Left Reference Location
Left Edge of Deck
Left Load Distance from Left Ref. Location
0.
Right Reference Location
Left Edge of Deck
Right Load Distance from Right Ref. Location
1.
Load Vertical Location
Top Slab is Loaded at Midheight of its Thinnest Portion
OK
Cancel

Figure A.22

A.1.6 Bridge object data is in Figure A.23. Distances from start abutment to span 1 is 130, span 2 is 323, and to span 3 462.

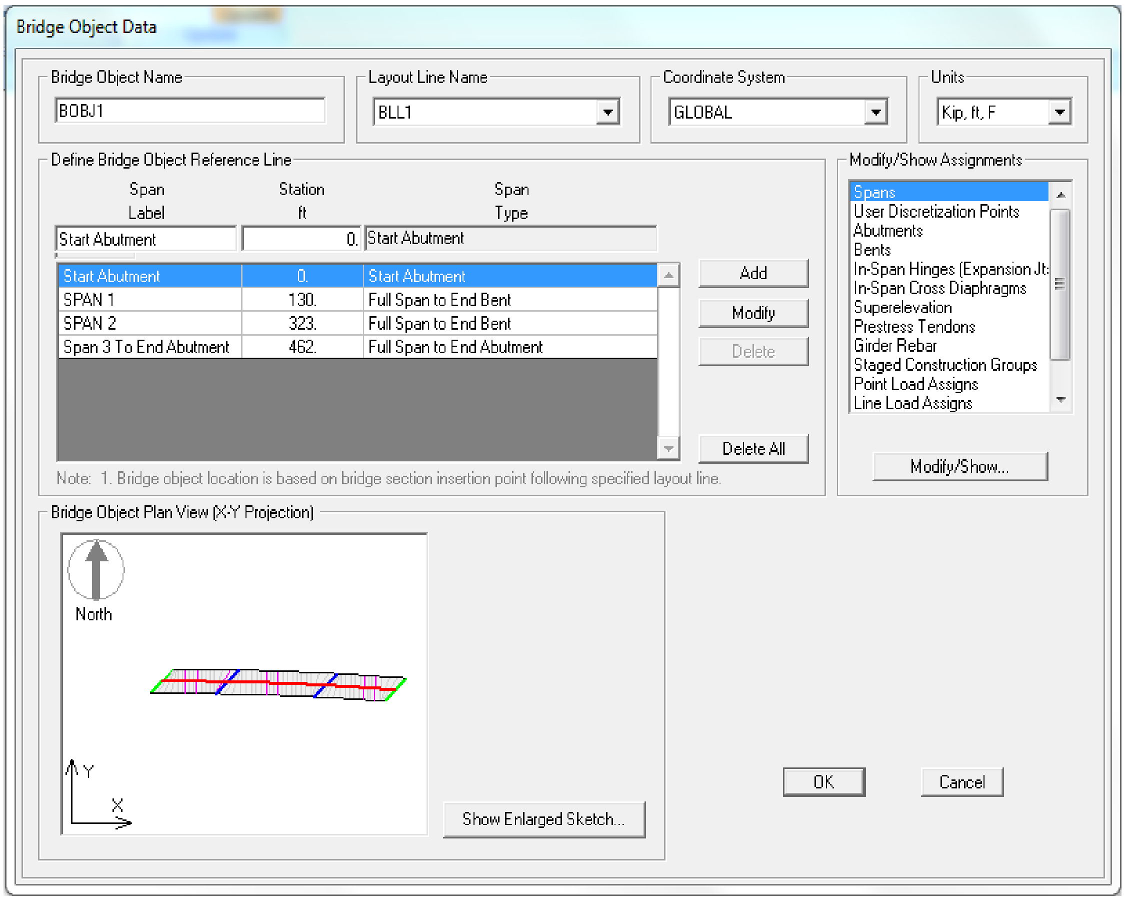

Figure A.23

A.1.7 Analysis

Dead load moment diagram shown in Figure A.24 is one of diagrams that obtained by CSI Bridge analysis.

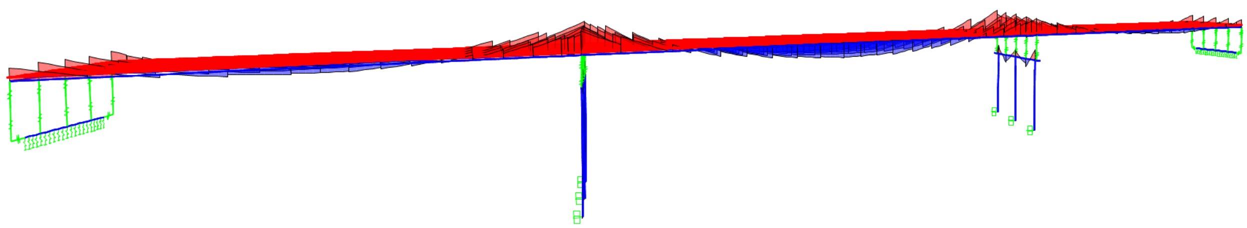

Figure A.24 Dead load Moment Diagram

Appendix B

B.1 Survey Instrument

Investigation of Closure Pour Elimination for Phased Construction of Steel Girder Bridges

Welcome to the NDOR (M324) study on *Phased Construction of Steel Girder Bridges*. The purpose of this project is to address whether there is a need for closure pour phase to connect the two phases of construction of steel girder bridges. Differential elevation has been an issue in Nebraska that requires this procedure. This survey serves as a means of information collection to determine if other Departments of Transportation (DOTs) are having similar issues and how they handle the problem. Please be assured that the answers you provide will remain confidential. Details about confidentiality and other questions can be answered by Dr. Terri Norton, Assistant Professor of Construction (402-554-2564; tnorton3@unl.edu).

This study has been reviewed by the University of Nebraska-Lincoln Institutional Review Board (IRB). If you have questions about your rights as a participant in this study, then you may contact them at 402-472-6965.

We truly appreciate your time and help with this important study. This questionnaire should take about 15 minutes to complete.

<u>DESIGN:</u>

1. **Do you currently work on or have you previously worked a project related to Phased Bridge Construction?**

   Yes ☐ No ☐

2. **Do you leave the cross-frames and diaphragms loose between each phase until after all deck pours are complete?**

   Yes ☐ No ☐

3. **Do you include a closure pour (a pour phase that connects Phase 1 and Phase 2) as part of your procedures?**

   Yes ☐ No ☐

4. **If yes, what justifies the need for a closure pour (i.e. dead load deflection exceeds 2 in)?**

   Deflection ☐

   Span Length ☐

Other ________________ ☐

5. What is the width range for the closure pour?

12 - 24 inches ☐
24 – 48 inches ☐
48 – 60 inches ☐
60 – 72 inches ☐
72+ ☐

6. Is the location and width of the closure pour a function of the girder spacing?

Yes ☐ No ☐

7. Do you make the overhangs on the phase line girder and the exterior girder symmetrical?

Yes ☐ No ☐

8. Do you adjust your short term composite factor for deflection calculations?

Yes ☐ No ☐

CONSTRUCTION:

9. Is a paving machine required for the closure pour?

Yes ☐ No ☐

10. How/Where do you support the deck finishing machine during the phase 2 pour?

Completely on Phase 2 ☐
Partially on Phase 1 and Phase 2 ☐

11. Is a sealant used to seal the joints of the projects?

Yes ☐ No ☐

12. Did any of your projects have issues with differential elevation between phases?

Yes ☐ No ☐

If yes, please comment on those issues. ________________________________

____

____

____

13. What steps were taken to remediate the problem of differential elevation? (i.e. adding temporary barriers or equipment for additional load)

Temporary concrete barriers (additional load) ☐

Construction equipment (additional load) ☐

Temporary support ☐

Inter-phase jacking ☐

Other ________________ ☐

14. Were there other issues during the phased construction project? Please explain.

____

____

____

____

<u>MONITORING</u>:

15. Have sensors and monitoring equipment (surveying, tolerance, etc.) been used to assess the performance of one of your phased constructed bridge? Please explain.

Yes ☐ No ☐

If yes, please comment on those issues. ______________________________

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B.2 Regional Differences

Q1. Do you currently work on or have you previously worked on a project related to Phased Bridge Construction?

Region	Yes	No
Northeast	4	0
Southeast	5	0
Southwest	3	1
West	3	0
Midwest	6	0

Q2. Do you leave the cross-frames and diaphragms loose between each phase until after all deck pours are complete?

Region	Yes	No
Northeast	3	1
Southeast	4	1
Southwest	1	3
West	3	0
Midwest	5	1

Q3. Do you include a closure pour (a pour phase that connects Phase 1 and Phase 2) as part of your procedures?

Region	Yes	No
Northeast	3	1
Southeast	4	1
Southwest	1	3
West	3	0
Midwest	5	1

Q4. If yes, what justifies the need for a closure pour (i.e. dead load deflection exceeds 2 in)?

Region	Deflection	Other	No Answer
Northeast	2	2	0
Southeast	2	2	1
Southwest	1	1	2
West	2	1	0
Midwest	3	3	0

Q4O: Other

Region	Response
Northeast	“Closure pour is preferred to reduce exposure to vibrations from adjacent stage 1 traffic.”
	“We typically have the longitudinal deck joint between the stages over a beam.”
Southeast	“Georgia uses closure pours only for continuous steel bridges that are constructed under traffic. For simple spans constructed under traffic, closure pours are not used.”
	“Required on steel girder bridges”
Southwest	“Phase construction issues are always taken on a case-by-case basis. Cross frames haven been temporarily left out, or they have been installed with slotted connection holes, all with varying degrees of success. Closure pours are employed when the deflecti…”
West	“We don't have a set criteria. It is a project by project discussion.”
Midwest	“A closure pour is considered at a longitudinal construction joint, on a case-by-case basis, if either of the following conditions applies.  1) The bridge deck will deflect more than 2 inches (50 mm) under dead load. 2) The staged bridge co…”
	“Differential dead load deflection between phase construction exceeding 1/4".
	“Michigan typically does not require a longitudinal closure pour, however, we've been forced to on past deck replacement or superstructure replacements on curved and super elevated structures. Eliminating the parabolic curve in the deck, without changing t…”

Q5. What is the width range for the closure pour?

Region	24-48 inches	48-60 inches	72+ inches	No answer
Northeast	2	0	1	1
Southeast	3	0	1	1
Southwest	0	0	2	2
West	2	0	1	0
Midwest	3	3	0	0

Q6. Is the location and width of the closure pour a function of the girder spacing?

Region	Yes	No	No answer
Northeast	2	2	0
Southeast	0	4	1

Southwest	1	1	2
West	1	2	0
Midwest	3	3	0

Q7. Do you make the overhangs on the phase line girder and the exterior girder symmetrical?

Region	**Yes**	**No**	**No answer**
Northeast	3	1	0
Southeast	1	3	1
Southwest	1	2	1
West	0	3	0
Midwest	2	4	0

Q8. Do you adjust your short term composite factor for deflection calculations?

Region	**Yes**	**No**	**No answer**
Northeast	1	3	0
Southeast	1	3	1
Southwest	0	3	1
West	2	1	0
Midwest	0	6	0

Q9. Is a paving machine required for the closure pour?

Region	**Yes**	**No**	**No answer**
Northeast	0	4	0
Southeast	0	4	1
Southwest	1	2	1
West	0	3	0
Midwest	0	6	0

Q10. How/Where do you support the deck finishing machine during the phase 2 pour?

Region	**Completely on Phase 2**	**Partially on Phase 1 and Phase 2**	**No answer**
Northeast	1	2	1
Southeast	0	3	2
Southwest	1	2	1
West	3	0	0
Midwest	3	3	0

Q11. Is a sealant used to seal the joints of the projects?

Region	Yes	No	No answer
Northeast	1	3	0
Southeast	0	4	1
Southwest	2	2	0
West	0	3	0
Midwest	2	4	0

Q12. Did any of your projects have issues with differential elevation between phases?

Region	Yes	No
Northeast	3	1
Southeast	3	2
Southwest	1	3
West	2	1
Midwest	3	3

Q12O: Comment on issues

Region	Comment on issues
Northeast	“Our most recent issue involved a 9-span 1600' long bridge (max span = 275') built in phases. 0 degree skew. Lack of symmetrical overhangs, as well as the Contractor's placement of concrete barrier prior to placement of the closure pour, caused differentia…”
	“the deflections of phase 2 did not equal phase 1 so the closure pour had a significant slope, which was in a wheel line”
	“Usually with curved or skewed bridges. Those type structures require a more thorough analysis in design.”
Southeast	“Issues with camber and with cross (transverse) slope of bridge deck.”
	“Phase two did not deflect the total amount show in the design calculations creating a rise instead of a fall in the bridge deck between phase one and phase two”
	“The cross frames could not be loosely bolted before the Phase 2 deck pour nor completely bolted after the Phase 2 deck pour.”
Southwest	“This is the typical issue when the phase construction joint is over a beam. This beam deflects half the amount of adjacent beams and then doesn't deflect further when the next phase of deck is placed.”
West	“Calculated dead load deflection exceed what was seen in the field causing a grade break at the phase line.”
	“Our larger projects on major river crossings. These issues are worked out by the design build team. I am not up to speed on the details of those issues and solutions.”

Midwest	“As mentioned earlier, curved super elevated bridges constructed in stages typically have grade challenges between stages 1 and 2 that may require a closure pour. Otherwise none is specified.”
	“Isolated incidents that were addressed by surface grinding”
	“The fabricator did not understand/follow the contract plan details of slotted holes.”

Q13. What steps were taken to remediate the problem of differential elevation? (i.e. adding temporary barriers or equipment for additional load)

Region	**Temporary concrete barriers (additional load)**	**Construction equipment (additional load)**	**Other**	**No answer**
Northeast	0	1	2	1
Southeast	1	1	2	1
Southwest	0	0	1	3
West	0	0	3	0
Midwest	2	0	0	4

Q13O. Other

Region	**Response**
Northeast	“Considered temporary concrete barriers to help correct the rotation but decided to live with the cross-slope deviation.”
	“It was noticed after the concrete placement. We performed some grinding of the deck.”
Southeast	“1. Allowed holes in one end of cross frame to be omitted and then field drilled after the Phase 2 pour. Advised Contractor that temporary timber bracing wedged between the beams/girders could be used during the deck pour.”
	“The difference in the required vs actual elevation for phase two was not severe. Grinding the completed bridge deck removed the regions which were too high.”
Southwest	“Lowering the bearing seat elevations of second phase beams.”
West	“Additional load of some kind”
	“Adjusted haunches”
	“The asphalt overlay place on the structure was used to smooth out the grade break.”

Q14. Were there other issues during the phased construction project? Please explain.

Region	**Yes**	**No**	**No answer**
Northeast	1	3	0

Southeast	1	4	0
Southwest	0	4	0
West	0	3	0
Midwest	2	3	1

Q14O

Region	Response
Northeast	“Forms set incorrectly that resulted in excessive deck overhang deflection.”
Southeast	“1. Allowed the elimination of tying the reinforcing steel between Phase 1 and Phase 2. 2. Advised the Contractor that provisions for differential elevations should be addressed for the permanent steel deck forms.”
Midwest	“It can be difficult to get lap spliced transverse steel to slide past each other during deflections.”
	“We often experience phased construction issues on prestressed concrete beams with camber growth.”

Q15. Have sensors and monitoring equipment (surveying, tolerance, ect.) been used to assess the performance of one of your phased constructed bridge? Please explain.

Region	Yes	No
Northeast	0	4
Southeast	0	5
Southwest	0	4
West	0	3
Midwest	0	6

QDC Do you have any additional comments related to design?

Region	Yes	No
Northeast	4	0
Southeast	2	3
Southwest	1	3
West	1	2
Midwest	3	3

QDCO

Region	Response
Northeast	“Be aware of placing non-composite loads (barriers) prior to placing the closure pour.”

	"If it cannot be made wide enough, mechanical connectors shall be utilized on the transverse reinforcement. Consideration should also be given to increasing its width to keep the first and/or second stage overhang from becoming too large."
	"The Maryland State Highway Administration currently has a study underway by the University of Maryland on closure pours. We've experienced problems in the past so we are trying to develop better parameters for their successful use."
	"We typically place the longitudinal deck joint between different stages at a girder line. We typically do not specify a true closure pour. A closure pour would be needed if using precast deck panels."
Southeast	"Minimum closure pour width is 2'"
	"The following is a link to NC's Bridge Design Manual. Section 6.2.2.8 discusses closure pours and longitudinal joints in bridge decks https://connect.ncdot.gov/resources/Structures/StructureResources/LRFD_Manual_Text_2012.pdf"
Southwest	"In some cases, we ask for survey of the phase I construction joint after the deck is poured to verify deflections and any adjustments in grade that may be necessary."
West	"We require the Contractor for submitting the deck overhang calculations during the deck pour."
Midwest	"Illinois has been studying our staged construction bracing for straight and skewed beams and we plan to issue revised policies in the next few months. The revised policy will not cover curved girders. We always encourage that every effort should be mad…"
	"see http://www.iowadot.gov/bridge/policy/52DecklrfdJa13.pdf for more information"
	"These items are typically dealt with on a case by case basis, as we have no guidance in our design specifications."

QCC Do you have any additional comments related to Construction?

Region	**Yes**	**No**
Northeast	0	0
Southeast	0	5
Southwest	1	3
West	1	2
Midwest	2	4

QCCO

Region	**Response**
Southwest	"We ask for survey of the phase I construction joint after the deck is poured to verify deflections and any adjustments in grade that may be necessary."
West	"The reinforcing lap splice length needs to be increased by 20% at the closer pour to accommodate for the live load deflection on phase 1."
Midwest	"We do not have a standard practice or policy for closure pours yet. Tentative guidance is 2" of differential dead load deflection. Some projects depending on geometry is difficult to have access to a closure pour area."

	"We have attempted to specify slotted holes and combinations of slotted holes in the past but we have discovered that slotted holes don't perform well because they bind up. This leads to thin decks. Sometimes they move but in a delayed fashion after the…"

QMC Do you have any additional comments related to Monitoring?

Region	Yes	No
Northeast	1	3
Southeast	0	5
Southwest	0	4
West	1	2
Midwest	0	6

QMCO

Region	Response
Northeast	"We are considering using monitoring in the future."
West	"We monitor using visual surveys during our normal bridge inspections."

Appendix C

C.1 Sensor Measurement and Conversions

EL Tilt Sensor No. 17480 at Girder A

Polynomial Factors

(Range of +/- 0.688 degrees)

C5	-0.012459887
C4	0.306900116
C3	-2.98766266
C2	14.3573956
C1	-34.3133631
C0	33.218176

Linear Factors

(Range of +/- 0.1146 degrees)

m	-0.283674569
b	1.42097726

Deviation (Linear Factors) = mX + b

Change	=	Current - Initial

Gauge length of sensor is 0.9144m

Deviation (Poly Factors)= $C_5X^5+C_4X^4+C_3X^3+C_2X^2+C_1X+C_0$ (mm/m)

		Deviation					**Deviation**	
		Poly	***Change***	**Linear**			**Poly**	***Change***
Date	EL Reading	in.	in.	in.	Date	EL Reading	in.	in.
14-May	4.54544	0.0050		0.0047	8-Jun	4.54380	0.0050	0.0000
15-May	4.55229	0.0049	-0.0001	0.0047	9-Jun	4.54720	0.0050	0.0000
16-May	4.57124	0.0047	-0.0003	0.0045	10-Jun	4.55090	0.0049	-0.0001
17-May	4.58991	0.0045	-0.0005	0.0043	11-Jun	4.54300	0.0050	0.0000
18-May	4.57471	0.0047	-0.0003	0.0044	12-Jun	4.54340	0.0050	0.0000
19-May	4.57403	0.0047	-0.0003	0.0044	13-Jun	4.54610	0.0050	0.0000
20-May	4.56419	0.0048	-0.0002	0.0045	14-Jun	4.52310	0.0052	0.0002
21-May	4.54730	0.0049	0.0000	0.0047	15-Jun	4.53670	0.0051	0.0001
22-May	4.55984	0.0048	-0.0001	0.0046	16-Jun	4.53550	0.0051	0.0001
23-May	4.55548	0.0049	-0.0001	0.0046	17-Jun	4.52870	0.0051	0.0002
24-May	4.55375	0.0049	-0.0001	0.0047	18-Jun	4.52860	0.0051	0.0002
25-May	4.55935	0.0048	-0.0001	0.0046	19-Jun	4.55540	0.0049	-0.0001
26-May	4.54834	0.0049	0.0000	0.0047	20-Jun	4.53430	0.0051	0.0001
27-May	4.54700	0.0050	0.0000	0.0047	21-Jun	4.54730	0.0049	0.0000
28-May	4.55470	0.0049	-0.0001	0.0046	22-Jun	4.53740	0.0051	0.0001
29-May	4.56150	0.0048	-0.0002	0.0046	23-Jun	4.53820	0.0050	0.0001
30-May	4.55680	0.0049	-0.0001	0.0046	24-Jun			
31-May	4.53760	0.0050	0.0001	0.0048				
1-Jun	4.54020	0.0050	0.0001	0.0048				
2-Jun	4.54530	0.0050	0.0000	0.0047				
3-Jun	4.53890	0.0050	0.0001	0.0048				
4-Jun	4.54280	0.0050	0.0000	0.0048				
5-Jun	4.54680	0.0050	0.0000	0.0047				
6-Jun	4.54530	0.0050	0.0000	0.0047				
7-Jun	4.53820	0.0050	0.0001	0.0048				

EL Tilt Sensor No. 17478 at Girder B

Polynomial Factors

(Range of +/- 0.688 degrees)

C5	-0.014448033
C4	0.360975154
C3	-3.56896389
C2	17.4484908
C1	-42.501005
C0	41.9562291

Linear Factors

(Range of +/- 0.1146 degrees)

m	-0.336099428
b	1.68287224

Deviation (Linear Factors) = mX + b

Gauge length of sensor is 0.9144m

Deviation (Poly Factors)= $C_5X^5+C_4X^4+C_3X^3+C_2X^2+C_1X+C_0$ (mm/m)

		Deviation					**Deviation**	
		Poly	***Change***	**Linear**			**Poly**	***Change***
Date	EL Reading	in.	in.	in.	Date	EL Reading	in.	in.
14-May	4.76861	0.00299		0.00289	8-Jun	4.7480	0.00325	0.00026
15-May	4.76493	0.00304	0.00005	0.00293	9-Jun	4.7431	0.00331	0.00032
16-May	4.76152	0.00308	0.00009	0.00297	10-Jun	4.7483	0.00324	0.00025
17-May	4.76834	0.00299	0.00000	0.00289	11-Jun	4.7503	0.00322	0.00023
18-May	4.76410	0.00305	0.00006	0.00294	12-Jun	4.7489	0.00324	0.00024
19-May	4.75888	0.00311	0.00012	0.00300	13-Jun	4.7501	0.00322	0.00023
20-May	4.75431	0.00317	0.00018	0.00306	14-Jun	4.7417	0.00333	0.00033
21-May	4.75300	0.00319	0.00019	0.00307	15-Jun	4.7509	0.00321	0.00022
22-May	4.75456	0.00317	0.00017	0.00306	16-Jun	4.7477	0.00325	0.00026
23-May	4.74805	0.00325	0.00025	0.00313	17-Jun	4.7455	0.00328	0.00029
24-May	4.75456	0.00317	0.00017	0.00306	18-Jun	4.7439	0.00330	0.00031
25-May	4.74780	0.00325	0.00026	0.00314	19-Jun	4.7439	0.00330	0.00031
26-May	4.75363	0.00318	0.00019	0.00307	20-Jun	4.7547	0.00316	0.00017
27-May	4.76490	0.00304	0.00005	0.00293	21-Jun	4.7486	0.00324	0.00025
28-May	4.75910	0.00311	0.00012	0.00300	22-Jun	4.7453	0.00328	0.00029
29-May	4.76110	0.00308	0.00009	0.00298	23-Jun	4.7448	0.00329	0.00030
30-May	4.75470	0.00316	0.00017	0.00305	24-Jun			
31-May	4.75200	0.00320	0.00021	0.00309				
1-Jun	4.75810	0.00312	0.00013	0.00301				
2-Jun	4.75430	0.00317	0.00018	0.00306				
3-Jun	4.75630	0.00314	0.00015	0.00303				
4-Jun	4.75560	0.00315	0.00016	0.00304				
5-Jun	4.75520	0.00316	0.00017	0.00305				
6-Jun	4.75810	0.00312	0.00013	0.00301				
7-Jun	4.75900	0.00311	0.00012	0.00300				

EL Tilt Sensor No. 17487 at Girder C

Polynomial Factors

(Range of +/- 0.688 degrees)

C5	-0.010574235
C4	0.253511498
C3	-2.39709662
C2	11.1656501
C1	-25.8879504
C0	24.5340273

Linear Factors

(Range of +/- 0.1146 degrees)

m	-0.290940677
b	1.45587016

Deviation (Linear Factors) = mX + b

Gauge length of sensor is 0.9144m

Deviation (Poly Factors)= $C_5X^5+C_4X^4+C_3X^3+C_2X^2+C_1X+C_0$ (mm/m)

		Deviation					Deviation	
		Poly	*Change*	Linear			Poly	*Change*
Date	EL Reading	in.	in.	in.	Date	EL Reading	in.	in.
14-May	4.8364	0.00175		0.00176	8-Jun	4.7913	0.00224	0.00050
15-May	4.8368	0.00174	0.00000	0.00175	9-Jun	4.7885	0.00227	0.00053
16-May	4.8523	0.00157	-0.00018	0.00159	10-Jun	4.7853	0.00231	0.00056
17-May	4.8455	0.00165	-0.00010	0.00166	11-Jun	4.7876	0.00228	0.00054
18-May	4.8413	0.00169	-0.00005	0.00170	12-Jun	4.7769	0.00240	0.00065
19-May	4.8231	0.00189	0.00015	0.00189	13-Jun	4.7903	0.00225	0.00051
20-May	4.7949	0.00220	0.00046	0.00219	14-Jun	4.7681	0.00250	0.00075
21-May	4.7985	0.00216	0.00042	0.00215	15-Jun	4.7712	0.00246	0.00072
22-May	4.8028	0.00211	0.00037	0.00211	16-Jun	4.7714	0.00246	0.00072
23-May	4.8013	0.00213	0.00039	0.00212	17-Jun	4.7739	0.00243	0.00069
24-May	4.8026	0.00212	0.00037	0.00211	18-Jun	4.7642	0.00254	0.00079
25-May	4.8005	0.00214	0.00039	0.00213	19-Jun	4.7649	0.00253	0.00079
26-May	4.7976	0.00217	0.00043	0.00216	20-Jun	4.7811	0.00235	0.00061
27-May	4.8111	0.00202	0.00028	0.00202	21-Jun	4.7773	0.00240	0.00065
28-May	4.8080	0.00206	0.00031	0.00205	22-Jun	4.7628	0.00256	0.00081
29-May	4.8107	0.00203	0.00028	0.00202	23-Jun	4.7686	0.00249	0.00075
30-May	4.7993	0.00215	0.00041	0.00214	24-Jun			
31-May	4.7993	0.00215	0.00041	0.00214				
1-Jun	4.8030	0.00211	0.00037	0.00211				
2-Jun	4.7969	0.00218	0.00043	0.00217				
3-Jun	4.8030	0.00211	0.00037	0.00211				
4-Jun	4.7948	0.00220	0.00046	0.00219				
5-Jun	4.7958	0.00219	0.00045	0.00218				
6-Jun	4.7926	0.00223	0.00048	0.00221				
7-Jun	4.7955	0.00220	0.00045	0.00218				

EL Tilt Sensor No. 17485 at Girder D

Polynomial Factors

(Range of +/- 0.688 degrees)

C5	-0.00431634
C4	0.108928599
C3	-1.07928797
C2	5.24721647
C1	-12.8172776
C0	13.2218425

Linear Factors

(Range of +/- 0.1146 degrees)

m	-0.301523688
b	1.50465054

$$\text{Deviation (Linear Factors)} = mX + b$$

Gauge length of sensor is 0.9144m

$$\text{Deviation (Poly Factors)}= C_5X^5+C_4X^4+C_3X^3+C_2X^2+C_1X+C_0 \quad \text{(mm/m)}$$

		Deviation					Deviation	
		Poly	*Change*	Linear			Poly	*Change*
Date	EL Reading	in.	in.	in.	Date	EL Reading	in.	in.
14-May	5.01636	-0.00031		-0.00028	8-Jun	4.9533	0.00041	0.00072
15-May	5.03724	-0.00054	-0.00024	-0.00051	9-Jun	4.9452	0.00050	0.00081
16-May	5.04898	-0.00068	-0.00037	-0.00064	10-Jun	4.9462	0.00049	0.00080
17-May	5.05050	-0.00069	-0.00039	-0.00066	11-Jun	4.9476	0.00048	0.00078
18-May	5.04841	-0.00067	-0.00036	-0.00063	12-Jun	4.9635	0.00030	0.00060
19-May	5.03483	-0.00052	-0.00021	-0.00048	13-Jun	4.9774	0.00014	0.00044
20-May	4.96113	0.00032	0.00063	0.00032	14-Jun	4.9621	0.00031	0.00062
21-May	4.98547	0.00005	0.00035	0.00005	15-Jun	4.9449	0.00051	0.00081
22-May	4.99966	-0.00012	0.00019	-0.00010	16-Jun	4.9438	0.00052	0.00083
23-May	4.97228	0.00020	0.00050	0.00019	17-Jun	4.9355	0.00061	0.00092
24-May	4.97160	0.00020	0.00051	0.00020	18-Jun	4.9321	0.00065	0.00096
25-May	4.94861	0.00047	0.00077	0.00045	19-Jun	4.9311	0.00066	0.00097
26-May	4.94153	0.00055	0.00085	0.00053	20-Jun	4.9581	0.00036	0.00066
27-May	4.95570	0.00038	0.00069	0.00037	21-Jun	4.9517	0.00043	0.00074
28-May	4.94930	0.00046	0.00076	0.00044	22-Jun	4.9313	0.00066	0.00097
29-May	4.95330	0.00041	0.00072	0.00040	23-Jun	4.9294	0.00068	0.00099
30-May	4.94930	0.00046	0.00076	0.00044	24-Jun			
31-May	4.94800	0.00047	0.00078	0.00046				
1-Jun	4.96350	0.00030	0.00060	0.00029				
2-Jun	4.95170	0.00043	0.00074	0.00042				
3-Jun	4.95170	0.00043	0.00074	0.00042				
4-Jun	4.93880	0.00058	0.00088	0.00056				
5-Jun	4.94360	0.00052	0.00083	0.00051				
6-Jun	4.95740	0.00037	0.00067	0.00036				
7-Jun	4.95780	0.00036	0.00067	0.00035				

EL Tilt Sensor No. 17482 at Girder E

Polynomial Factors

(Range of +/- 0.688 degrees)

C5	-0.026735412
C4	0.65832875
C3	-6.4272536
C2	31.0827803
C1	-74.7241293
C0	72.0559284

Linear Factors

(Range of +/- 0.1146 degrees)

m	-0.305486404
b	1.53175822

Deviation (Linear Factors) = mX + b

Gauge length of sensor is 0.9144m

Deviation (Poly Factors)= $C_5X^5+C_4X^4+C_3X^3+C_2X^2+C_1X+C_0$ (mm/m)

		Deviation					**Deviation**	
		Poly	***Change***	**Linear**			**Poly**	***Change***
Date	EL Reading	in.	in.	in.	Date	EL Reading	in.	in.
14-May	5.11299	-0.00112		-0.00109	8-Jun	5.1165	-0.00116	-0.00004
15-May	5.12050	-0.00121	-0.00009	-0.00117	9-Jun	5.1183	-0.00118	-0.00006
16-May	5.11848	-0.00118	-0.00006	-0.00115	10-Jun	5.1275	-0.00129	-0.00017
17-May	5.11984	-0.00120	-0.00008	-0.00116	11-Jun	5.1378	-0.00141	-0.00029
18-May	5.11623	-0.00116	-0.00004	-0.00112	12-Jun	5.1294	-0.00131	-0.00019
19-May	5.13455	-0.00137	-0.00025	-0.00132	13-Jun	5.1475	-0.00152	-0.00040
20-May	5.12170	-0.00122	-0.00010	-0.00118	14-Jun	5.1424	-0.00146	-0.00034
21-May	5.13185	-0.00134	-0.00022	-0.00129	15-Jun	5.1383	-0.00141	-0.00029
22-May	5.12307	-0.00124	-0.00012	-0.00120	16-Jun	5.1352	-0.00138	-0.00026
23-May	5.12449	-0.00125	-0.00013	-0.00121	17-Jun	5.1339	-0.00136	-0.00024
24-May	5.11802	-0.00118	-0.00006	-0.00114	18-Jun	5.1328	-0.00135	-0.00023
25-May	5.11840	-0.00118	-0.00006	-0.00115	19-Jun	5.1315	-0.00133	-0.00021
26-May	5.12012	-0.00120	-0.00008	-0.00117	20-Jun	5.1369	-0.00140	-0.00028
27-May	5.12100	-0.00121	-0.00009	-0.00117	21-Jun	5.1345	-0.00137	-0.00025
28-May	5.12010	-0.00120	-0.00008	-0.00117	22-Jun	5.1256	-0.00127	-0.00015
29-May	5.11630	-0.00116	-0.00004	-0.00112	23-Jun	5.1325	-0.00134	-0.00023
30-May	5.11710	-0.00117	-0.00005	-0.00113	24-Jun			
31-May	5.11560	-0.00115	-0.00003	-0.00112				
1-Jun	5.11730	-0.00117	-0.00005	-0.00113				
2-Jun	5.12550	-0.00126	-0.00014	-0.00122				
3-Jun	5.11770	-0.00117	-0.00005	-0.00114				
4-Jun	5.12550	-0.00126	-0.00014	-0.00122				
5-Jun	5.11090	-0.00110	0.00002	-0.00106				
6-Jun	5.10950	-0.00108	0.00004	-0.00105				
7-Jun	5.10890	-0.00107	0.00005	-0.00104				

EL Tilt Sensor No. 17486 at Girder F

Polynomial Factors

(Range of +/- 0.688 degrees)

C5	-0.002803021
C4	0.068697398
C3	-0.656895099
C2	3.05733986
C1	-7.17925117
C0	7.39663123

Linear Factors

(Range of +/- 0.1146 degrees)

m	-0.274800165
b	1.37325236

Deviation (Linear Factors) $= \mathrm{mX} + \mathrm{b}$

Gauge length of sensor is 0.9144m

Deviation (Poly Factors)= $C_5X^5+C_4X^4+C_3X^3+C_2X^2+C_1X+C_0$ (mm/m)

		Deviation					Deviation	
		Poly	*Change*	Linear			Poly	*Change*
Date	EL Reading	in.	in.	in.	Date	EL Reading	in.	in.
14-May	-	-	-		8-Jun	10.0020	-0.32047	-0.00009
15-May	-	-	-		9-Jun	10.0020	-0.32047	-0.00009
16-May	-	-	-		10-Jun	10.0020	-0.32047	-0.00009
17-May	-	-	-		11-Jun	10.0024	-0.32060	-0.00022
18-May	-	-	-		12-Jun	10.0017	-0.32038	0.00000
19-May	-	-	-		13-Jun	10.0020	-0.32047	-0.00009
20-May	-	-	-		14-Jun	10.0020	-0.32047	-0.00009
21-May	-	-	-		15-Jun	10.0020	-0.32047	-0.00009
22-May	-	-	-		16-Jun	10.0014	-0.32029	0.00009
23-May	10.0017	-0.32038	-	-0.04951	17-Jun	10.0025	-0.32063	-0.00025
24-May	10.0020	-0.32048	-0.00010	-0.04951	18-Jun	10.0027	-0.32069	-0.00031
25-May	10.0014	-0.32028	0.00010	-0.04950	19-Jun	10.0027	-0.32069	-0.00031
26-May	10.0014	-0.32028	0.00010	-0.04950	20-Jun	10.0025	-0.32063	-0.00025
27-May	10.0014	-0.32029	0.00009	-0.04950	21-Jun	10.0020	-0.32047	-0.00009
28-May	10.0020	-0.32047	-0.00009	-0.04951	22-Jun	10.0020	-0.32047	-0.00009
29-May	10.0020	-0.32047	-0.00009	-0.04951	23-Jun	10.0014	-0.32029	0.00009
30-May	10.0020	-0.32047	-0.00009	-0.04951	24-Jun			
31-May	10.0020	-0.32047	-0.00009	-0.04951				
1-Jun	10.0020	-0.32047	-0.00009	-0.04951				
2-Jun	10.0020	-0.32047	-0.00009	-0.04951				
3-Jun	10.0014	-0.32029	0.00009	-0.04950				
4-Jun	10.0020	-0.32047	-0.00009	-0.04951				
5-Jun	10.0014	-0.32029	0.00009	-0.04950				
6-Jun	10.0020	-0.32047	-0.00009	-0.04951				
7-Jun	10.0020	-0.32047	-0.00009	-0.04951				

EL Tilt Sensor No. 17484 at Girder G

Polynomial Factors
(Range of +/- 0.688 degrees)

C5	-0.003368475
C4	0.086997704
C3	-0.88046616
C2	4.36031012
C1	-10.8639817
C0	11.5278798

Linear Factors
(Range of +/- 0.1146 degrees)

m	-0.315786941
b	1.58110395

Deviation (Linear Factors) $= mX + b$

Gauge length of sensor is 0.9144m

Deviation (Poly Factors)= $C_5X^5+C_4X^4+C_3X^3+C_2X^2+C_1X+C_0$ (mm/m)

		Deviation					**Deviation**	
		Poly	***Change***	**Linear**			**Poly**	***Change***
Date	EL Reading	in.	in.	in.	Date	EL Reading	in.	in.
14-May	-	-	-	-	8-Jun	10.0020	-0.30362	-0.0001
15-May	-	-	-	-	9-Jun	10.0020	-0.30362	-0.0001
16-May	-	-	-	-	10-Jun	10.0020	-0.30362	-0.0001
17-May	-	-	-	-	11-Jun	10.0024	-0.30374	-0.0002
18-May	-	-	-	-	12-Jun	10.0017	-0.30354	0.0000
19-May	-	-	-	-	13-Jun	10.0020	-0.30362	-0.0001
20-May	-	-	-	-	14-Jun	10.0020	-0.30362	-0.0001
21-May	-	-	-	-	15-Jun	10.0020	-0.30362	-0.0001
22-May	-	-	-	-	16-Jun	10.0014	-0.30345	0.0001
23-May	10.0017	-0.30353	-	-0.05678	17-Jun	10.0025	-0.30377	-0.0002
24-May	10.0020	-0.30363	-0.00010	-0.05679	18-Jun	10.0020	-0.30362	-0.0001
25-May	10.0014	-0.30343	0.00010	-0.05678	19-Jun	10.0020	-0.30362	-0.0001
26-May	10.0020	-0.30363	-0.00010	-0.05679	20-Jun	10.0025	-0.30377	-0.0002
27-May	10.0014	-0.30345	0.00009	-0.05678	21-Jun	10.0020	-0.30362	-0.0001
28-May	10.0020	-0.30362	-0.00009	-0.05679	22-Jun	10.0020	-0.30362	-0.0001
29-May	10.0014	-0.30345	0.00009	-0.05678	23-Jun	10.0014	-0.30345	0.0001
30-May	10.0020	-0.30362	-0.00009	-0.05679	24-Jun			
31-May	10.0020	-0.30362	-0.00009	-0.05679				
1-Jun	10.0020	-0.30362	-0.00009	-0.05679				
2-Jun	10.0020	-0.30362	-0.00009	-0.05679				
3-Jun	10.0014	-0.30345	0.00009	-0.05678				
4-Jun	10.0020	-0.30362	-0.00009	-0.05679				
5-Jun	10.0014	-0.30345	0.00009	-0.05678				
6-Jun	10.0020	-0.30362	-0.00009	-0.05679				
7-Jun	10.0014	-0.30345	0.00009	-0.05678				